CITY OF
FORT LAUDERDALE

Memorandum

Memorandum No: 25-054

TO: Honorable Mayor and Members of the Fort Lauderdale City Commission

FROM: Rickelle Williams, City Manager RW

DATE: May 5, 2025

SUBJECT: **New Police Headquarters — Phase 2 Peer Review**

This memorandum is being sent to update the City Commission on the Phase 2 peer review being conducted by Wiss, Janey, Elstner Associates, Inc. (WJE) related to certain portions of the Fort Lauderdale Police Department Headquarters (PDHQ).

Background

As you are already aware, an issue became apparent during the installation of glazing on the new PDHQ, which resulted in the need to have third-party engineering professionals perform peer reviews on the structure and provide analysis and feedback on remedies and structural enhancements.

During a scheduled meeting on March 25, 2024, City staff was notified by AECOM Technical Services, Inc. (the Architect of Record) and Thornton Tomasetti, Inc. (the Engineer of Record) of a structural crack along the northern portion of the roof slab of the new PDHQ. The issue stemmed from a design miscalculation involving the roof's cantilever that manifested as a result of the precast panels. Although initially considered minor, immediate actions were taken including revised permits, structural inspections, and a temporary stop work order in the affected area while work in other locations of the project continued uninterrupted. Subsequently, the City retained Lakdas/Yohalem Engineering, Inc. as a third-party consultant to validate and assess the original repairs. Its report, dated May 31, 2024, determined that the prior corrective measures were insufficient and recommended additional structural reinforcement. Please see Memo 24-091 (Attachment 1) for more information on previous actions.

However, it was determined that an additional peer review should be performed to validate the enhancements related to corrective measures to resolve the deflection issue. Per the direction of the City Commission, staff conducted interviews with engineering firms and determined that WJE would be the best firm to perform this additional peer review. In addition, the City Commission directed WJE to perform a peer review of the remainder of the building.

To address the issue of the deflection, WJE proposed to complete the peer review in two (2) phases. Phase 1 would focus on the deflection area analysis and subsequent recommendations to resolve the issue. Phase 2 would focus on an analysis of the remainder of the building by providing findings and recommendations for addressing the findings. On December 31, 2024, WJE provided the City with the results of the Phase 1 report which resulted in additional structural enhancements to columns and support beams, and foundation work with micro piles. These recommendations have been presented to AECOM and both AECOM and WJE have come to a consensus on actions needed to implement these enhancements. This work is now being implemented by Moss & Associates, LLC (Moss), the project contractor, through the City's permitting process. The City Attorney's Office has prepared a draft amendment to the AECOM agreement that intends to hold AECOM responsible for all costs associated with the deflection and extend the monitoring period from approximately five (5) years to fifteen (15) years. The draft amendment will be shared with AECOM this week.

On April 22, 2025, WJE provided the City with a draft Phase 2 peer review. This review analyzes the remaining portions of the building for structural deficiencies, provides code references in support of the findings, and makes conceptual recommendations for addressing these issues.

The draft WJE Phase 2 peer review has identified critical findings that include life-safety concerns requiring timely corrective action. These findings span key structural elements such as shear wall and foundation capacities, slide bearing capacity, roof beam capacities, and deficiencies in column axial-flexural and shear capacities.

WJE also noted documentation gaps and calculation concerns related to the building's structural behavior and design coordination—such as discrepancies in column schedules, mechanical equipment loads, masonry wall support, and column flexural stiffness modifiers, among other concerns. The City is prioritizing review of these critical findings and working closely with the full project team to determine and implement appropriate engineering responses. Please see draft WJE Peer Review, Phase 2 – Building Structural Design Peer Review for more detailed information (Attachment 2).

As this process progresses, the following steps are anticipated moving forward:

- The draft Phase 2 report will be provided to Moss for awareness;
- The draft Phase 2 report will be transmitted to AECOM and the Engineer of Record, requesting a response from them within 14 calendar days; and
- The AECOM response will be reviewed by the City and WJE to evaluate next steps.

As always, safety, transparency, and accountability remain the City's highest priorities, and additional updates will be provided as solutions are finalized. In the coming days City staff will have follow-up meetings with both WJE and AECOM to develop recommendations intended for implementation. It is the City's expectation that any validated deficiencies will be included in the Errors and Omissions section of the contract with AECOM.

At the May 20, 2025, City Commission meeting, WJE will provide a review of its draft Phase 2 peer review findings and respond to any questions.

You may contact Assistant City Manager Anthony Fajardo at 954-828-5758 or via email at afajardo@fortlauderdale.gov should you have any questions or concerns.

Attachments:

1. Commission Memo 24-091–New Police Headquarters - Roof Deck Cracking/Deflection
2. Draft WJE Peer Review, Phase 2 – Building Structural Design Peer Review

c: D'Wayne M. Spence, Interim City Attorney
David R. Soloman, City Clerk
Patrick Reilly, City Auditor
City Manager's Office
Department Directors

CITY OF
FORT LAUDERDALE

Memorandum

Memorandum No: 24-091

Date: June 14, 2024

To: Honorable Mayor, Vice Mayor, and Commissioners

From: Susan Grant, Acting City Manager

Subject: New Police Headquarters – Roof Deck Cracking/Deflection

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This memo is being sent to update the City Commission on a construction issue that became apparent during the installation of glazing on the new Police Department Headquarters (PDHQ).

On March 25, 2024, as part of a regularly scheduled meeting, AECOM staff along with engineering firm Thornton Tomasetti (the Engineer of Record) informed the City, verbally, of a structural crack that was forming along the roof slab on the northwest corner of the PDHQ Building as a result of continuous deflection and bending moment. Thorton Tomasetti stated that this was due to a structural design error in the calculations for this area. The roof cantilever was causing stress and deflection and after the precast panels were installed, a crack started to occur. Basically speaking, the support structures were incorrectly designed in relation to the weight of the cantilevered roof area above the third floor of the building.

This crack referenced is obvious to the naked eye and continuous along the entire length of the beam (both sides) as evidenced by documentation on file with the Building Services Division. A drawing showing a detail of a potential correction for this condition was also presented by Thorton Tomasetti to City staff during this meeting. These drawings proposed repair work involving additional rebar and concrete for the northward expansion of the third-floor columns. Along with this, the ground floor footings were redesigned to incorporate an additional rebar cage to account for the additional loads created by the expansion of the third-floor columns.

This issue was of great concern to City staff present on the call who were assured that this was minor in nature and posed no harm to the building and/or future construction work in this area.

Immediately following the meeting, City staff met internally to discuss this issue with the Development Services Department's (DSD) Building Services Division on how best to address this issue. The initial response was communicated to the development team of Moss & Associates, AECOM, and Thornton Tomasetti to implement the following initial remedial steps to begin to address this issue:

1. Immediate written notification to the Contractor and City Building Division (structural), of any shoring and/or bracing that is being installed to prevent any further damage and maintain life safety.

2. Submission of a revised building permit set to be filed with the Building Services Division of DSD, prior to any structural repair work being proposed.

3. Immediate inspection by the Building Services structural reviewers and inspectors until a remedy is agreed upon.

4. No continuation of any work in the general area that might add to the already stressed load causing additional deterioration of the structural integrity of this portion of the roof slab.

5. No further repair work without obtaining the proper Building Services reviews and approvals.

6. A written letter by the structural engineer stating a description of this deficiency in detail and a thorough re-examination of the balance of the building's structural calculations and written findings and confirmation that no other beams and/or components are found to have similar deficiencies.

7. All Threshold Inspection Reports and Material Testing Reports submitted immediately to the Building Services Division as required by the Florida Building Code.

Due to the urgent, emergency nature of the work, AECOM, along with Thornton Tomasetti developed the initial corrective measure which Moss & Associates implemented. A permit revision was requested by the Building Servies Division and approved on March 27, 2024.

The Building Services Division continued monitoring the deflection reports of the referenced beam submitted by Thorton Tomasetti and informed senior level staff that the beam was in fact continuing to deflect at 1/100th of a foot over a five-day period. This is after the proposed repair work had been completed and after the complete concrete cure time frame period of 28 days.

As a result of a city inspection, the City placed a partial stop work order on the project for this area. This was done to allow the City time to engage a third-party structural engineer to review this condition and make recommendations on whether a more suitable repair solution should be implemented. It is important to note that work is permitted to proceed in other areas of the construction site as there has been no indication this issue exists elsewhere on the site.

On May 13, 2024, the City engaged Lakdas/Yohalem Engineering, Inc as a third-party structural engineering firm to validate the repair and offer additional recommendations if needed. In the report from Lakdas/Yohalem Engineering, Inc dated May 31, 2024, recommendations were made to include additional structural repairs as the repairs implemented by Thornton Tomasetti were deemed inadequate to fully address the deflection of the beam.

Currently, the City's development team is fully engaged with Moss & Associates, AECOM, Thorton Tomasetti, and Lakdas/Yohalem Engineering on moving forward with implementation of the additional structural repairs. These include additional temporary structural support in specified areas, along with engineered structural solutions to supplement those already implemented by Moss & Associates as specified by Thorton Tomasetti.

At this time all parties are collaborating on the final work that needs to occur to have this addressed. AECOM and Moss & Associates have been very cooperative with the third-party engineer and are working with City staff and the engineer of record (Thornton Tomasetti) to move forward.

The deflection of the roof slab in this area does not pose an immediate threat to the structural integrity of the building, however, from what has been explained by the City's Building Services team and various engineers working on the project, this deflection does have the potential to cause issues after the building is completed and occupied. These may include the displacement of glazing allowing for water intrusion and additional structural repairs needed as time passes. This may impact warranty claims and other forms of remedies if not suitably addressed at this time.

Based on the continued discussions with the third-party engineering firm and the development team of Moss & Associates and AECOM we are optimistic that a suitable solution can be implemented to address this issue. As more information becomes available, we will make sure to update the City Commission.

You may contact Assistant City Manager Anthony Fajardo at 954-828-5758 or via email at afajardo@fortlauderdale.gov should you have any questions or concerns.

c: Anthony G. Fajardo, Assistant City Manager
Laura Reece, Acting Assistant City Manager
Ben Rogers, Acting Assistant City Manager
Thomas J. Ansbro, City Attorney
David R. Soloman, City Clerk
Patrick Reilly, City Auditor
Department Directors
CMO Managers

Fort Lauderdale Police Headquarters

WJE Peer Review, Phase 2 – Building Structural Design Peer Review

1300 West Broward Boulevard
Fort Lauderdale, Florida 33312

Draft Report

April 22, 2025
WJE No. 2024.4855.0

PREPARED FOR:
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PREPARED BY:
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Draft Report

Fort Lauderdale Police Headquarters

WJE Peer Review, Phase 2 – Building Structural Design Peer Review

1300 West Broward Boulevard
Fort Lauderdale, Florida 33312

Draft Report

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Draft Report

Zack Coleman, PhD
Project Associate

April 17, 2025
WJE No. 2024.4855.0

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Draft Report

Draft Report

1. INTRODUCTION

At the request of the City of Fort Lauderdale (the City), Wiss, Janney, Elstner Associates, Inc. (WJE) has completed our Phase 2 – Building Structural Design Peer Review for the new Fort Lauderdale Police Headquarters (FLPHQ) building, located at 1300 West Broward Boulevard, Fort Lauderdale, Florida, 33312. This report summarizes the scope of our review, the project background, and our document review. It also presents the findings from our structural design review of selected portions of the structure, accompanying discussion, and additional comments and recommendations for consideration.

2. SUMMARY OF FINDINGS

Findings of our structural design review of the FLPHQ building are summarized below. Only comments which warrant direct response from Thorton Tomasetti (TT), the Structural Engineer of Record (SEOR), are provided below. Items 1 through 8 are related to life safety and warrant attention either *as soon as possible* or in the near term. Suggested timeframes to complete rectifications are provided for each life-safety finding. Time frames are not provided for serviceability (Items 9 through 11) or documentation findings (Items 12 through 16), but we recommend that these items be resolved as soon as is practicable, but no later than within 12 months.

More information about our approach, discussion, and, as-appropriate, conceptual rectification measures are provided within the body of the report. Extensive discussion regarding the portion of structure at the north elevation—including information about observed deflection at the cantilevered spans and structural rectifications in place—is provided in our previously submitted Phase 1 report[1]. Our findings below are limited to the portion of building not examined in Phase 1—i.e., everything south of Gridline 2. The SEOR should address our comments below and make the necessary rectifications to the structure.

Life Safety

1. **Shear Wall Capacities.** The southern-most reinforced concrete core wall is not code-compliant for one-way shear strength or flexural strength between Levels 1 and 2. Adding a properly designed full-height shear wall (i.e., from foundation to roof) with an accompanying foundation to the structure on Gridline 14 between Gridlines H and J can rectify this condition. Rectification of this condition should be carried out <u>*as soon as possible*</u> (0 to 2 months) and should be completed before the building is occupied. Note that, contrary to the position of the SEOR, we do *not* believe that the concrete masonry unit (CMU) partition walls are a reliable part of the lateral load-resisting system. We have provided other findings related to the use of these CMU walls in the Documentation section below.
2. **Foundation Capacities at Shear Walls.** The mat foundations for the concrete shear walls around the north stair shaft (designated as CF-1 on the structural drawings) and the south elevator shaft (CF-3) all have lateral demands that exceed the allowable design capacity under design wind loads in the east-west direction and are therefore not code-compliant. Adding the aforementioned shear wall to the building can rectify conditions related to the south elevator shaft but not the sliding exceedance at the north stair shaft. Attaching mat footing CF-1 to its surrounding isolated

[1] WJE. December 31, 2024. *Fort Lauderdale Police Headquarters. WJE Peer Review, Phase 1 – North Elevation Evaluation.*

footings can rectify the sliding condition. Rectifications to the foundations should take place at the same time as those for the shear walls.

3. **Foundation Capacities at Columns.** The bearing stress under the three isolated footings on Gridline 4, between Gridlines G and K, exceed the allowable bearing capacity of 7,000 psf and are therefore not code-compliant. This condition can be rectified by enlarging these footings. These rectifications should take place before the building is occupied (0 to 6 months).
4. **Slide Bearing Capacity.** The bearing stress in the slide bearings on Gridline F (adjoining the community room and lobby area to the main structure of the FLPHQ) exceeds the manufacturer's published allowable capacity when subjected to design loads. Since the design demand exceeds the manufacturer's published design capacity, these bearings are not code-compliant. This condition can be rectified by replacing the existing bearings with a higher-capacity system. Replacement of the bearings may also require rectifications at the top of the stub column below the bearing to accommodate the size and thickness of the new bearing. This rectification should be completed before the building is occupied (0 to 6 months). We also recommend that the SEOR comments on the ranges of temperature and exposure conditions which the slide bearings will be subjected to and how those ranges will affect bearing performance. The SEOR should also provide the peak lateral displacement expected across the slide bearing under factored design level loads and/or thermal temperature differences for review.
5. **Structural Integrity.** It appears that the design used a noncontact lap splice of prestressing strand to "shear friction bars" to address code-requirements for continuity of bottom reinforcement in soffit beams. This type of detail is not addressed in either ACI 318-14[2] or the Florida Building Code (FBC)[3], and we are not aware of any other documentation that explicitly allows this type of detail. We acknowledge that this building system is widely used in the South Florida market. However, the SEOR should provide calculations, results of load tests, or other valid engineering documentation which demonstrate that this type of connection is adequate to develop continuity of bottom reinforcement prior to occupancy of the building.
6. **Column Axial-Flexural Capacities.** Based on our analysis, 24 columns were overstressed[4] for combined axial and flexural loading and thus are not strictly code-compliant. Only 8 of those columns were overstressed by more than 10%. While the FLPHQ is not considered an existing building (but rather new design), codes for existing buildings, including the 2020 Florida Building Code, Existing, 7th Edition (FEBC) include allowances for the gravity load-carrying structural elements (e.g., columns) to undergo up to a 5% increase in design gravity load without being strengthened. Additionally, the actual in-place concrete and steel material strengths are likely higher than the nominal design values used for capacity calculations. Further, there is the

[2] ACI 318-14: Building Code Requirements for Structural Concrete

[3] 2020 Florida Building Code, Building, 7th Edition (FBC)

[4] The extent to which a member satisfied code-required strength levels was quantified using the ratio of design demand to design capacity, (i.e., demand-to-capacity ratio or DCR). A DCR greater than 1.0, i.e., unity, indicates that the demand is greater than the capacity, meaning the element is overstressed, while a DCR less than 1.0 indicates that the design demands are less than the design capacity.

possibility of moment redistribution once a structural member inelastically deforms and loses stiffness in indeterminate structures such as this building. Therefore, we believe that the 16 columns that are overstressed less than 10% would likely perform as intended without any strengthening. Nonetheless and in light of the noncompliance, considerations other than safety may be applicable to the City's decision regarding strengthening of these columns. Rectifications to the 8 columns that were overstressed by more than 10% should take place before the building is occupied (0 to 6 months).

7. **Column Shear Capacities.** Furthermore, 23 columns were found to be overstressed in one-way shear, and 50 were found to require minimum transverse reinforcement spaced no more widely than one-half of the effective depth. For 48 of those 50 columns, the shear demand exceeded one-half of the reduced nominal concrete shear strength (the limit beyond which minimum reinforcement must be provided) by more than 10%. Both conditions are related to an ACI 318-14 code requirement predicated on the understanding that transverse reinforcement spaced more widely than one-half of the effective section depth may not intercept a shear crack[5]. However, the transverse reinforcement in these columns is only slightly more widely spaced than one-half of the effective depth. Furthermore, the axial compression in the columns is likely to result in shear cracks that are more closely aligned with the column axis, allowing the cracks to intercept some transverse reinforcement before propagating though the section. Finally, there exist technical documents and codes which indicate that transverse reinforcement spaced somewhat more widely than one-half of the effective depth will intercept shear cracks and enhance shear capacity. Therefore, while the columns with either shear condition do not strictly satisfy ACI 318-14, we believe they will still perform adequately and do *not* require shear strengthening to achieve structural safety. Nonetheless and in light of the noncompliance, considerations other than safety may be applicable to the City's decision regarding strengthening of these columns. Any rectifications to columns should take place before the building is occupied (0 to 6 months).
8. **Roof Beam Capacities.** Several beams adjacent to the mechanical equipment on the roof do not have code-required strength for flexure, one-way shear, and combined shear and torsion limit states when the 150 pounds per square foot (psf) mechanical live load allowance stated on the drawings (Sheet HQ-S0-2-02) is applied to the building. If however, the weight of the equipment (Sheets HQ-S2-2-4C and HQ-S2-2-4D) was explicitly used to design these beams in lieu of the 150 psf live load allowance, it is possible that the beams would have minimum required strength. Section 1603.1.1 Floor Live Load of the FBC states that "The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas." Currently there is a discrepancy or lack of clarity on the structural design drawings for the load that was used in the design. The SEOR should revisit the design of these beams to confirm they have sufficient capacity for the intended loading. The intended loading should be clarified on the structural drawings. Any necessary rectifications should take place before this area is fully loaded, or within 12 months.

[5] It is expected that reinforced concrete members (beams, columns, walls, etc.) will crack prior to, and typically well before, reaching their ultimate capacity. When we write of concrete cracking throughout this report in the context of design, note that we are indicating normal and expected behavior of reinforced concrete members.

Serviceability

9. **Community Room and Lobby Area (Area E).** The roof drift of the community room and lobby area (designated as "Area E" on the structural drawings) are highly sensitive to the assumed fixity of the isolated footings. Depending on the assumed fixity and the location in the building, the drifts may or may not satisfy the recommended drift limits in ASCE/SEI 7-16[6], which aim to minimize wind-related serviceability issues. The SEOR should clarify what assumption regarding the footing fixity (e.g., pinned or fixed boundary conditions) was used when designing the FLPHQ. Related to this topic, the SEOR should clarify what value for the modulus of subgrade reaction was used for the design, provide accompanying justification from the geotechnical engineer, and report the value on the structural drawings. Additionally, for the systems that are impacted by building drift (e.g., window wall and finishes), the architect-of-record (AOR) or the design professional in charge of the system should review the design of the system to confirm that the system is compatible with the estimated building drifts.
10. **Building Period.** Due to the relatively small length of shear wall in the southern half of the main building of the FLPHQ, the fundamental period of vibration of the building is over 0.9 seconds (which is significantly larger than typical concrete shear wall buildings having no more than three stories). The fundamental mode also corresponds with a twisting response of the structure. While this condition does not violate any code-prescribed performance limitations, it is not standard practice to design structures to perform this way. Adding an appropriately designed full-height shear wall (i.e., from foundation to roof) to the structure (as previously described) will rectify this condition.
11. **Cover Depth.** The precast concrete shop drawings inconsistently indicate the amount of cover depth provided in the soffit beams. In some cases, the cover depth is indicated to satisfy minimum code cover requirements, and in others it does not. The SEOR should confirm and document which cover depth was used in the design of the soffit beams.

Documentation

12. **Column Schedule.** The column schedule and plan views in the structural drawings use conflicting gridlines. The former refers to Gridline K.8, which does not exist on the plan view. We recommend that the structural drawings are updated to rectify this inconsistency.
13. **Loads.** The structural drawings do not indicate that rain loading or flood loading was incorporated into the design. However, the plumbing drawings do show primary and secondary drainage systems on the roof. Our understanding is that, at the time of design of the FLPHQ, the site was within Zone X of the 2014 FEMA floor maps, meaning that design for flood loads would not have been required. Nonetheless, it should be clarified on the structural drawings if, and how, each type of load was considered.
14. **Mechanical Equipment.** In addition to the loading discrepancy noted in Item 8 above regarding equipment weight versus uniform live loads, there is an area bounded by Gridlines H, J, 14, and 16 on Level 2 that is designated in the architectural drawings, but not in the structural drawings, as

[6] ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Structures

containing mechanical equipment. Similar to Item 8, the intended use of this area should be clarified on the drawings and the applicable loads for this area should be confirmed and clarified on the drawings.

15. **Masonry Walls.** We do not believe that the original design intent of the FLPHQ was to use the typical concrete masonry unit (CMU) partition walls as shear walls as the SEOR has recently indicated. The structural drawings, specifications, and special inspection reports provide a wealth of information (as summarized below) demonstrating that the masonry walls were not originally intended or designed to be a part of the lateral force-resisting system.

    Sheet HQ-S0-1-01 describes the lateral system as "Ordinary Reinforced Concrete Shear Walls", making no mention of masonry walls.

    Sheet HQ-S0-1-01 also notes that the seismic response modification factor used was 4.0, rather than 2.0, which would have been required if ordinary reinforced masonry shear walls were used as part of the lateral system.

    On numerous sheets in the structural drawings (e.g., Sheet HQ-S2-2-1A), the CMU partition walls are referred to as "non-loadbearing CMU" which bear on thickened slab built over a vapor retarder (Detail 7 of Sheet HQ-S7-1-02). A vapor retarder would reduce the sliding resistance of the wall.

    The masonry walls are not listed in the "S3 Series Drawings" which are described in the structural drawings as containing "lateral system elevations, connection forces and details." These sections, including the shear wall schedule, refer to the cast-in-place concrete shear walls (consistent with Sheet HQ-S0-1-01), and make no mention of masonry.

    None of the connections of the partition masonry walls (i.e., not including the bearing wall in the community room and the masonry core wall in the main structure) to the diaphragm shown in the structural drawings (Detail 6 of Sheet HQ-S7-1-02) or noted in the special inspection reports (i.e., PTA anchors) satisfy ACI 318-14 Section 12.5.3.7. In other words, these connections do not satisfy the shear-friction provisions of ACI 318, which invalidates the masonry walls as code-compliant contributors to lateral resistance.

    Furthermore, the alternate option provided in Detail 6 of Sheet HQ-S7-1-02 shows a minimum gap of 1 inch between the top of the walls and the bottom of the above floor. Should this detail have been used, the walls would only start resisting floor loading once (or if) the gap closed, clearly indicating that the SEOR did not originally intend for the walls to carry axial loading from the above floor. In their recent lateral system calculations package (dated almost three years after issuance of the permitted structural drawings), the SEOR indicated that axial load in the walls was requisite to develop the sliding resistance of the so-called "CMU shear walls." It is confounding that the SEOR would propose using a connection detail that effectively precludes axial load from entering the wall (apart from self-weight) if their design relied on sliding resistance, unless of course, the original design did not intend for those walls to be a part of the lateral system of the structure.

    Based on our review of the manufacturer's website, the PTA anchors do not appear to have a load rating for either seismic or wind loads that would allow an engineer to design the shear transfer at the slab-wall interface. The lack of a quantifiable shear transfer capacity at these interfaces makes it also appear like these walls could not have been an engineered part of a lateral system.

Unlike the partition walls, the masonry wall in Area E was reasonably designed and constructed to act as a shear wall. That wall a) was constructed to be in bearing and support roof loads, b) was detailed to have fully developed shear friction reinforcement (as required by ACI 318-14), and c) had a standalone wall footing without a vapor retarder underneath it. The contrast between the design of this wall and the typical masonry walls underscores the conclusion that the typical masonry walls were not originally intended or designed to be part of the lateral system.

While we do not believe that the CMU partition walls were originally intended or designed to be a part of the lateral system of the FLPHQ, they will participate in the resistance of at least some lateral load by virtue of being present. This participation is typically neglected in building designs. Nonetheless, even given the SEOR's current contention that the walls were intended to be a part of the original lateral system, we believe that the SEOR's approach to evaluating the lateral system is also non-compelling to demonstrate that the CMU walls are adequate for at least the following additional reasons:

Some of the stiffness modifiers used to determine the load distribution between the reinforced concrete and masonry walls and set forth in the calculation package provided by the SEOR are unrealistic, do not reflect the expected behavior of reinforced concrete members, and result in artificially low estimates of the amount of shear demand in a CMU wall. Given the low tensile strength of concrete, the relatively low axial load in the walls, and the SEOR's calculations that indicate that the demand in at least one concrete wall segment may exceed its design capacity, it is implausible that the concrete walls would be uncracked at factored loading. If increased flexibility of the reinforced concrete walls was properly accounted for, the demand in the CMU walls would likely increase beyond their design capacity (as reported by the SEOR).

The effects of staged construction were not considered when determining the axial load in the CMU walls, resulting in an erroneously high estimate of the sliding resistance under the walls.

A vapor retarder was used under the slab on ground which would reduce the frictional resistance under the slab. It is not clear that the effects of the vapor retarder are reflected in the SEOR's understanding of the sliding resistance under the walls. The SEOR should provide documentation from the geotechnical engineer that clearly justifies the use of a frictional coefficient of 0.30 for soil beneath a vapor retarder.

For the reasons outlined above, we recommend that the SEOR reevaluate the adequacy of the FLPHQ without considering the partition masonry walls as part of the lateral force-resisting system. Note that WJE has *not* considered these partition masonry walls as contributing to the lateral force-resisting system when evaluating the FLPHQ and developing our findings. If the SEOR abandons the approach of attempting to use the partition masonry walls as part of the lateral force-resisting system, then adding the full-height shear wall and rectifying the foundations as noted above in Items 1 and 2, and properly designing them for predicted demands is expected to be sufficient to resolve Item 15.

During our review of the special inspection reports, we also observed that the PTA anchors used to connect the partition masonry walls to the above floors were not oriented orthogonal to the walls, as shown on the anchor manufacturer's website. Accordingly, we recommend that the SEOR comment on which orientation of the anchors is appropriate and provide documentation from the anchor manufacturer supporting the use of that orientation.

16. **Column Flexural Stiffness Modifiers.** The documents we received from the SEOR indicate that a wide range of column flexural stiffness modifiers were used in the design of the FLPHQ, ranging from assuming pin-ended behavior (i.e., effectively a modifier of 0.00) to using a modifier of 0.70. We concur that in the design of a building, it may be prudent to use different modifiers when designing (e.g.,) the gravity- and lateral-load resisting system. For example, ACI 318-14 Section 6.6.3.1.1 recommends using a flexural stiffness modifier of 0.70, but Section 6.6.3.1.2 allows a modifier of 0.50 to be used when considering lateral loads, thereby conservatively decreasing the lateral loads resisted by the columns and increasing those used to design the shear walls. However, the SEOR appears to be using at least three different stiffness modifiers (0.00, 0.35, and 0.70), whichever is most favorable to their position, regardless of whether or not it is legitimate or reasonably supported by code provisions. In fact, the SEOR's use of modifiers to obtain favorable results seems to be contrary to the intent of ACI 318-14. We recommend that the SEOR carefully review the impact of column stiffness modifiers to determine if their understanding of member demands is within accepted engineering practice. Supporting documentation (code language, research papers, etc.) should be provided to substantiate the use of stiffness modifiers which are contrary to code provisions. If the SEOR accepts the recommendations made by WJE in Items 1 through 7 above, it is not necessary to respond to this item and no further changes are expected.

3. PROJECT BACKGROUND

A description of the FLPHQ, summary of our previous work (described in detail in our Phase 1 report), and scope of work undertaken for this report are provided below.

3.1. Description of the Structure

The FLPHQ is a 191,000-square-foot, three-story structure currently under construction. The building will include workspace for over 700 personnel, including training rooms, public meeting areas, and a community space. The design of the building was led by AECOM (the prime consultant), with Thornton Tomasetti (TT) as the Structural Engineer-of-Record (SEOR). The general contractor for the Project is Moss and Associates (Moss). Collectively, these entities are referred to as the Project Team. TT also serves as the threshold inspector for the building.

The main portion of the building is rectangular in plan, with the length of the building oriented on a north-south axis. The north elevation of the building has a step at the third floor and roof, forming a cantilevered projection at these floors. An isometric view of the building is shown in Figure 1. The facade of the building is clad with a combination of precast concrete fascia and full-height glass fenestration systems. An overall plan view of the foundation level of the building and the gridlines is shown in Figure 2.

The gravity framing of the structure consists of one-way reinforced concrete slabs spanning between east-west oriented precast joists. These joists are supported by north-south oriented reinforced concrete soffit beams, which transfer load to the cast-in-place concrete columns.

The lateral force-resisting system for the building is composed of cast-in-place reinforced concrete shear walls. See Figure 2 for the locations (on plan) of these walls.

The foundations for the columns are spread footings bearing on soil in which vibro-compaction was used to improve the bearing strength. The foundations of the reinforced concrete core walls are shallow mat footings which all bear on improved soil.

3.2. Previous Work

On December 31, 2024, WJE issued a report regarding the findings of Phase 1 of our peer review of the FLPHQ. Our scope of work in Phase 1 consisted of assessing the extent and severity of issues with the original design of the north elevation of the building (north of Gridline 2). Investigation of these issues was motivated by the facts that the tip of the cantilevered roof beams deflected substantially (reportedly 1/2 to 3/4 inches) after attaching the precast facade panels to the structure, and there was cracking of the roof slab and roof cantilever beams. Various measures to rectify the cantilever beam deflections and other issues that the Project Team subsequently became aware of either were implemented or are planned to be implemented. Those measures are described in detail in our previous report. WJE made specific recommendations regarding the 1st story columns, 3rd story columns, and roof cantilever beams at Gridlines H and J; however, those recommendations will not be repeated here. Please refer to the Phase 1 report for further information.

3.3. Phase 2 Scope of Work

The objective of Phase 2 of WJE's scope of work was to provide a structural peer review of the remainder of the structure not covered in Phase 1—i.e., the portion of the FLPHQ south of Gridline 2. The scope of our work generally consisted of reviewing the design basis and the structural design of the FLPHQ, including the lateral system for the building as a whole. WJE also performed a site visit to facilitate our review of the structure.

The review of the design basis generally consisted of the following tasks:

1. Review the design documents to confirm that the design loads conform to the applicable codes.
2. Confirm that loads from major mechanical items are accommodated in the structural plans.
3. Review the structural design criteria and design assumptions for conformance to the applicable codes and generally accepted engineering practice.
4. Review the structural and architectural plans for the building and confirm that the structural plans generally conform with the architectural plans regarding loads and other conditions that may affect the structural design.
5. Review geotechnical reports and other engineering investigation reports that are related to the foundation and structural design, and confirm that the design documents properly incorporate the results and recommendations of the investigations.

The review of the structural design generally consisted of the following tasks:

1. Review the gravity and lateral-load paths of the building.
2. Perform calculations for a representative fraction of the systems, members, and details to check their adequacy to resist code required design demands.

3. Confirm that the structural integrity provisions of the applicable codes are being followed.

Note that it was not within our scope of work to check every element, connection, detail, condition, etc., in the building. While we used our experience and judgement to identify and check structural members that were either representative of typical members or that we believed to be critical, it is the ultimate responsibility of the Project Team to ensure that the as-built structure is in compliance with applicable codes and consistent with its original design intent.

4. DOCUMENT REVIEW

To complete our Phase 2 scope of work it was necessary to perform a document review supplemental to our Phase 1 review of the FLPHQ, although many of the documents reviewed for Phase 1 were re-visited for Phase 2. For brevity, the bulk of the re-visited documents will not be listed below. For this report, we have reviewed additional files which either a) have been developed since the issuance of our Phase 1 report, or b) were not applicable to the north elevation. While many of these documents relate to reply comments which we received from the SEOR regarding the conditions we identified in our Phase 1 report, it was instrumental to review those files for the purpose of our Phase 2 work as they offered insight into the design philosophy used by the SEOR which presumably governed the design of the remainder of the building not reviewed as part of Phase 1. The additional files reviewed are summarized below.

Geotechnical Report

- By Nutting Engineers
- Dated January 20, 2021
- Fifty sheets

Architectural Drawings

- By AECOM
- Dated June 10, 2022
- Two-hundred and eighty-two sheets

Plumbing Drawings

- By Hammond & Associates, Inc.
- With revisions through May 24, 2024
- Fifty-two sheets

Mechanical Drawings

- By AECOM
- With revisions through May 24, 2024
- Fifty-seven sheets

Electrical Drawings

- By AECOM
- With revisions through November 15, 2023
- Seventy-four sheets

TT Response to WJE Letter on Columns G/2 & K/2

- By Thornton Tomasetti
- File name is *20250217_FLPH Column G-2 and K-2 Validation.pdf*
- Dated February 17, 2025

- One sheet

Roof Beam RSB-76 & RSB-79 Validation of Torsional Capacity and Loading
- By Thornton Tomasetti
- File name is *2025311_FLPH Beam RSB-76 RSB-79 Validation.pdf*
- Dated March 11, 2025
- Three sheets

Columns G/2, K/2 Shear Capacity Validation
- By Thornton Tomasetti
- File name is *2025311_FLPH Column G-2 and K-2 2nd Validation.pdf*
- Dated March 11, 2025
- Three sheets

Columns H/2, J/2 Shear Capacity Validation
- By Thornton Tomasetti
- File name is *2025311_FLPH Column H-2 and J-2 Validation.pdf*
- Dated March 11, 2025
- Eight sheets

Lateral System Calculation Package
- By Thornton Tomasetti
- File name is *20250323_Police Headquarters Lateral System Calculations Package_2.pdf*
- Dated March 21, 2025
- Sixty-two sheets

4.1. Background on Stiffness Modifiers

This section provides background on the use of stiffness modifiers when developing numerical models for the design or analysis of a structure.

When reinforced, it is expected that concrete (and other quasi-brittle materials, such as masonry) will crack, allowing strain in the embedded reinforcement to develop the strength of the structural member. The cracking reduces the overall stiffness of the reinforced concrete member. In indeterminate structures such as the FLPHQ (in which there are many load paths through which external loading can be resisted), the distribution of forces in structural members is governed in part by the stiffnesses of those members, relative to one another. Therefore, if one member cracks, its stiffness decreases relative to the other members such that more force is distributed to the uncracked (i.e., stiffer) members. The increase in force of those other members may cause them to crack such that force is again redistributed throughout the structure. This process repeats until both a) the equilibrium of external forces with internal forces, and b) compatibility of deformations of the structure are satisfied.

It is seldom straightforward and practical to develop numerical models of structures that explicitly account for the reduction in member stiffness and redistribution of load caused by cracking in reinforced concrete. Accordingly, many designs rely on elastic analyses of the modeled structure that uses a single value of stiffness for a member, rather than accounting for the progressive increase in member flexibility due to cracking and inelastic deformation. Based on Section 6.6 of ACI 318-14 (which was used in the design of the FLPHQ, as discussed later), the value of stiffness is determined by modifying the cross-sectional properties of a member with a so-called stiffness modifier to reflect the degree of cracking and inelasticity expected at factored load levels. Tables 6.6.3.1.1(a–b) of ACI 318-14 set forth values of stiffness modifiers to achieve the reduced member stiffnesses that is to be used in elastic numerical models that support structural design.

In a numerical model of an indeterminate structure, the stiffness modifier applied to a member affects the amount of external loading which that member resists. For example, in a lateral wind-resisting system containing both reinforced concrete columns and shear walls, a reduction in the flexural stiffness modifier for the columns would decrease the overall lateral force resisted by the columns but increase the force resisted by the shear walls. Similarly, if the stiffness modifier for the columns was increased, the demands on the columns and walls would increase and decrease, respectively.

Given the variety of load combinations (and load effects) that a structure must be designed to resist, it is reasonable to use different sets of stiffness modifiers when designing different structural systems. This allows the designer to bound the range of load effects to conservatively design structural members. For example, when designing the gravity system of a structure, it is reasonable to model the columns using a flexural stiffness modifier of 0.70, resulting in relatively large flexural demands in the columns. However, when designing the lateral system, it may be more appropriate to use a reduced modifier (e.g., 0.50 as permitted by Section 6.6.3.1.2 of ACI 318-14) such that more of the total demand on the building is resisted by the primary lateral load-resisting element (e.g., shear walls in the FLPHQ) rather than the gravity elements (e.g., columns in the FLPHQ).

4.2. SEOR's Use of Column Stiffness Modifiers in Previous Documents

Given the importance of stiffness modifiers in bounding the loading demands on structural members in the FLPHQ, the SEOR's use of column stiffness modifiers in previous documents that we have received were reviewed. The following two documents pertain to conditions that we identified in our Phase 1 report, which needed to be reviewed by the SEOR for possible adaptation of the structural design of the FLPHQ. After developing our Phase 1 report, these conditions (among others) were discussed at several meetings between WJE, the SEOR, and the AOR. The following two documents were developed by the SEOR in response to WJE's position regarding the behavior of the structure (also summarized below).

4.2.1. SEOR's Response to WJE Letter on Columns G/2 & K/2

In response to our letter to the City (dated January 21, 2025), the SEOR issued a document (dated February 17, 2025) containing their opinion on the adequacy of the columns on Gridline 2, at the intersections of Gridlines G and K. For context, in our letter, WJE contended that the columns required minimum shear reinforcement to satisfy ACI 318-14, due largely to shear demands likely induced by end moments from the precast concrete floor joists, inducing torsion on the soffit beams. The SEOR contended that the joists were modeled as "pin-ended", and we disagreed that this was appropriate for the following reasons (among others).

> The shoring shop drawings and design specifications indicated that the structural framing was to be shored to the ground until the concrete placement integrates the joists, beam, and slab elements.

> Composite action can be achieved between the joists and slab since a) developed tension reinforcement was placed in the slab above the joists, b) the design specifications called for the top of the joists to be roughened to a 1/4-inch amplitude, and c) the joist ends were cast integrally with the beams on Gridlines G and K. Cazaly hangers (or other connections which would not allow for negative end moments to be developed) were not used.

In their response, the SEOR concluded that "Micro-cracking at the spandrel-column interface will relieve nearly all of the bending moment at the column tops at Columns G/2 & K/2, and thus the shear calculated by WJE will not enter the columns at all." To justify this conclusion, the SEOR approximated the flexural moment that would result in cracking of the columns, considering the tensile capacity of concrete and the compressive axial loading in the columns from gravity loading. The SEOR further noted that, based on WJE's calculations of the negative moment capacity of the composite joist and slab, the cracking resistance of the columns was expected to be surpassed and therefore, the columns would crack. Continuing, the SEOR claimed that "This is normal behavior in a structural floor system of this nature. The introduction of a very small flexural crack at the top and bottom of the column will cause the traditional pin-ended column behavior that is commonly assumed for this type of floor system." While we disagree with the SEOR that, once a reinforced concrete column cracks, it devolves into a pin-ended condition that no longer resists flexure at the cracked location, we understand this statement as indicating that the SEOR designed the FLPHQ by assuming that the columns have pinned ends. In other words, the columns had an effective flexural stiffness modifier of 0.00.

4.2.2. Columns H/2, J/2 Shear Capacity Validation

During our meetings with the SEOR, we also conveyed that we believed that the other columns on Gridline 2 (at the intersections of Gridlines H and J) required minimum transverse reinforcement due to the shear demand in the existing 16 inch by 16 inch columns. In their calculations package in response to our concern (dated March 11, 2025), the SEOR contended that, based on their numerical model, the columns had sufficient shear capacity such that Section 10.6.2.1 of ACI 318-14 did not require minimum transverse reinforcement for those columns. The SEOR provided column loads from their model (Figure 3) that were predicated on the use of a column flexural stiffness modifier of 0.35 to determine the loads. Using these demands, the SEOR calculated the nominal concrete shear strength using Equations 22.5.6.1(a–b). Since the bending moments in the columns were sufficiently small, Equation 22.5.6.1(b) was permitted to be used in lieu of Equation 22.5.6.1(a), the latter of which results in a lower calculated shear strength. Using Equation 22.5.6.1(b), the SEOR concluded that minimum transverse reinforcement was not required, based on Section 10.6.2.1 of ACI 318-14. While we still disagreed with the SEOR, we understand this calculation package as indicating that the SEOR designed the FLPHQ using a column flexural stiffness modifier of 0.35. This assumption is internally inconsistent with their previous claim that the members exhibit "pin-ended column behavior."

5. REVIEW OF DESIGN BASIS

The design basis for the building was reviewed to confirm that it was in accordance with the applicable codes, general engineering practice, and the other reports/documents prepared by the Project Team (e.g., the architectural drawings, the geotechnical report, etc.).

5.1. Applicable Codes, Loads, and Design Criteria Review

We reviewed the structural drawings and calculations provided by the SEOR to compare them with requirements of the codes governing the design of the FLPHQ. The structural drawings indicate that the following codes were used.

Building. 2020 Florida Building Code, Building, 7th Edition (FBC)

Structural Concrete. Building Code Requirements for Structural Concrete (ACI 318-14)

Structural Steel. Specification for Structural Steel Buildings (AISC 360-16)

Masonry. Building Code Requirements for Masonry Structures (TMS 402-16)

We also reviewed the structural drawings for background information regarding the various types of loads that structures are typically designed to resist, as summarized below.

Dead Loads. Normalweight concrete with a weight of 145 +/- 5 pcf (pounds per cubic foot) was specified. Normalweight concrete masonry unit (CMU) blocks with a weight of 135 pcf were specified. Sheets HQ-S0-2-01 through 02 indicate explicitly that the self-weight of the reinforced concrete slabs and precast joists was accommodated in the design of the structure. No mention is made of the precast facade panels. Superimposed dead loads (also shown on Sheets HQ-S0-2-01 through 02) ranged from 5 (at roof overhangs) to 40 psf (at occupied roof) depending on the location within the structure.

Live Loads. Live loads (shown on Sheets HQ-S0-2-01 through 02) ranged from 80 (in general locations) to 150 psf (at mechanical equipment locations). The roof live load was generally 30 psf, except at overhangs (i.e., on the flange of the precast façade panels) where the load was 20 psf.

Snow Loads. No snow load was used for the design of the structure.

Rain Loads. No design criteria were provided in the structural drawings for rain loads; however, primary and secondary drainage systems were shown on the plumbing drawings.

Flood Loads. No design criteria were provided in the structural drawings.

Wind Loads. The following design criteria were identified in the structural drawings as the basis for wind loads used in the design.

- Risk Category: IV
- Ultimate Wind Speed: 185 mph (miles per hour)
- Serviceability Wind Speed: 144 mph
- Exposure Category: C
- Internal Pressure Coefficient: +/- 0.18

Seismic Loads. The following design criteria were identified in the structural drawings as the basis for seismic loads used in the design.

- Seismic Importance Factor: 1.5
- Short-Period Spectral Response Acceleration Parameter: 0.050 g (gravitational acceleration)
- One-Second Spectral Response Acceleration Parameter: 0.022 g
- Site Class: D

- Seismic Design Category: A
- Lateral System Description: Ordinary Reinforced Concrete Shear Walls
- Seismic Response Coefficient: 0.017 g
- Response Modification Factor: 4

5.2. Design Drawings Review

Findings from our review of the structural drawings and architectural drawings are provided below.

5.2.1. Structural Drawings

We noted the following during our review of the structural drawings.

Masonry Walls. In their lateral system calculation package (dated March 21, 2025, nearly 3 years after the 100% construction documents were issued) covering the main portion of the FLPHQ (but not Area E on the plans), the SEOR claimed that the design intent for the FLPHQ was to utilize the masonry walls as shear walls. However, Sheets HQ-S3-1-01, -03, -04, and -05 (titled "Shear Wall Plans", "Shear Wall Schedules", "Shear Wall Details", and "Shear Wall Details", respectively) only showed reinforced concrete shear walls; no masonry shear walls were shown or mentioned. We did not observe anywhere on the permitted structural plans for the main portion of the FLPHQ that masonry walls were indicated as being a part of the lateral load-resisting system. Moreover, Sheet HQ-S0-1-01 explicitly defined the lateral system using standard code terminology as "Ordinary Reinforced Concrete Shear Walls" and provided a seismic response modification factor (R) of 4, which is consistent with an ordinary reinforced concrete shear wall system per Table 12.2-1 of ASCE/SEI 7-16, not with a masonry shear wall system. This demonstrates that the SEOR's original design of the lateral system relied entirely on reinforced concrete shear walls for lateral resistance because, if ordinary reinforced masonry shear walls were part of the lateral load-resisting system, a R value of 2 would have been required, per ASCE/SEI 7-16, which would have doubled the design seismic forces relative to what the SEOR says they used.

Masonry Wall Foundation Details. Note 4 on the foundation plans for the main portion of the FLPHQ (sheets HQ-S2-2-1A, HQ-S2-2-1B, HQ-S2-2-1C, HQ-S2-2-1D) states that "ALL INTERIOR NON-LOADBEARING CMU TO BEAR ON THICKENED SLAB, TYPICAL. SEE 7/S7-02". This note appears to be referring to Detail 7/HQ-S7-1-02, which is shown in Figure 4. Note that a vapor retarder passes underneath this thickened slab. If the interior CMU walls were to have a foundation other than this thickened slab, they would be called out with a WF# (i.e, wall footing) designation, and the dimensions of the wall footing would be shown in the wall footing schedule on sheet HQ-S6-1-01. We understand that if the WF# designation was provided for a CMU wall footing, then the detail for this footing would be 2/HQ-S6-1-03 (see Figure 5). Note that a vapor retarder does not pass under the wall footing in this detail. We did not observe any WF# designations on the plans under interior CMU walls for the main portion of the FLPHQ, except for the loading dock at Gridline L between Gridlines 10 and 12. The masonry shaft located just to the east of Gridline J and to the north of Gridline 14 has a wall designation of MW16. This shaft has its own foundation details, which are shown in 2/HQ-S6-1-01.

Masonry Wall Connection Details at Top and Bottom of Wall. Both 7/HQ-S7-1-02 (Figure 4) and 2/HQ-S6-1-01 (Figure 5) indicate that the vertical reinforcement at the bottom of the CMU walls should be lapped with dowels. These dowels were to extend into the footing, or thickened slab, and be hooked. At the top of the CMU walls, two details are provided on Sheet HQ-S7-1-02, which is labeled as "Typical Masonry Details". Detail 3/HQ-S7-1-02 appears to be a detail for use with CMU walls that are bearing walls. A note in this detail also indicates that this detail applies when tie-beams are referenced on plan. We did not observe any tie beam references on plan, and we also did not observe on the plans where any CMU walls were explicitly designated as bearing walls. As such, Detail 3/HQ-S7-1-02 appears to not be applicable to the interior masonry walls. A wall schedule shown on HQ-S6-1-01 (see Figure 6) also does not explicitly designate any of the CMU walls as bearing walls. The masonry shaft noted above would necessarily be a bearing wall as it supports the end of concrete joists, but all other walls (not within Area E) appear to be non-loadbearing walls and in fact are believed to have been constructed as infill after the concrete structure was built.

Detail 6/HQ-S7-1-02 (Figure 7) is further divided into two details, a typical detail that applies to both interior and exterior walls and an alternate option that applies to interior walls only. The typical detail shows the vertical reinforcement of the CMU wall embedded into the slab above. No minimum embedment depth is provided, but the details note that the slab thickness must be a minimum of 4 ¾ inches. Section 042200 of the specifications, "Concrete Masonry Units" Part 2.6.C addresses "Top of Wall Anchors." This specification section is shown in Figure 8. Interior non-load bearing partition walls in Seismic Design Category A, which applies to the FLPHQ, are shown to allow PTA series anchors by either Blok-Lok or by Hohmann & Barnard (H&B). A specification sheet for PTA 420 HS anchors by H&B is shown in Figure 9. A lateral load capacity for the anchor is not provided on this sheet nor is the type of fastener (e.g., screws) required to connect the anchor to the concrete structure above. It may be that this information exists somewhere else on H&B's website, but we could not find it. As such, it is unclear to us if these anchors are approved or rated for lateral load transfer, and if so, what load they are rated for. Also unclear is the capacity that the SEOR assumed for these anchors. A rendering of the PTA 420 HS anchor at the top of a CMU wall (also taken from H&B's website) is shown in Figure 10. Note that the anchor is shown with the length of the top plate oriented transverse to the length of the wall, suggesting that its rated capacity would be applicable to out-of-plane rather than in-plane loading.

Soil Properties. The permitted structural drawings did not indicate the coefficient of friction between the foundations and soil used in the design. However, in their lateral system calculation package, the SEOR claimed that they used a coefficient of friction of 0.3. The drawings also do not indicate what value was used for the modulus of subgrade reaction in the design.

Column Identification. The column schedule (Sheet HQ-S4-2-01) and plan views (e.g., Sheet HQ-S2-2-2A) use conflicting gridlines. The former references columns on Gridline K.8 which does not exist on the plan views.

Mechanical Equipment. The weight of some rooftop mechanical units is shown on Sheets HQ-S2-2-4C through 4D, but there is also a uniform live-load allowance shown on Sheets HQ-

S0-2-01 through 02 (both sheets are titled "Floor Loading Diagrams"). While it is presumed that the allowance on the latter sheets was used, this is not clear from the structural drawings.

Area E Lateral System. There is an expansion joint between Area E and the main portion of the FLPHQ that incorporates slide bearings. While this expansion joint allows vertical load to be transferred from the roof of the main building to columns in Area E, the slide bearings, discussed below, are intended to minimize the transfer of lateral load between Area E and the main building. Area E does not contain reinforced concrete shear walls. Based on the lack of reinforced concrete shear walls on the permitted structural drawings for Area E, it appears that the lateral load-resisting system for Area E is intended to be, in the east-west direction, a CMU masonry wall with reinforced concrete tie column boundary elements. In the north-south direction, a reinforced concrete moment resisting frame appears to be used. The aforementioned CMU wall is not denoted as a bearing wall on the plans; however, the section detail shown on the structural drawings (see Figure 11) indicates that vertical reinforcement from the CMU wall extends through the beam above it and into the slab further above, a distance of more than 2 feet. Additionally, the CMU is shown as being "keyed" or set into a recess in the bottom of the beam, implying that this particular CMU wall, in contrast to all other partition walls, was constructed before the concrete for the floor system above it was placed. Threshold inspection report No. 104 (see section on this report below) confirms that this particular wall was constructed prior to the roof structure. In contrast to the typical CMU walls, the strip footing designed for this wall is specified on Sheet HQ-S6-1-01 (dated June 10, 2022) as being 3 feet wide and 1 foot thick. Based on Detail 2/HQ-S6-1-03 (dated May 24, 2024) and Note 2 on sheet HQ-S2-2-1E, the top of the footing is set 2 feet below the finished slab elevation, and the vapor retarder is not continuous underneath the footing.

Slide Bearings. The SEOR has incorporated slide bearings into their design at the joint between Area E and the main portion of the building. These slide bearings are intended to allow the two buildings to move somewhat independently under lateral and thermal loading while transferring gravity loads across the joint. These slide bearings apply to columns at Grid Points F/3 and F/4. Figure 12 shows a section of the lower beam from Area E, a stub column extending up to the slide bearing, and the upper spandrel beam from the main building above the slide bearing. Figure 13 shows the detail for the slide bearing and calls out the specific slide bearing to be used, which is Con-Serv CSA slide bearing. Con-Serv's website indicates that the CSA bearings have an allowable bearing pressure of 2,000 pounds per square inch (psi) and also indicates that the total capacity of the bearing is based on the area of the lower portion of the bearing. Since the area of the lower portion of the bearing is 25 square inches (5 inches times 5 inches) in the design (see Figure 13), the allowable bearing capacity of the slide bearing is 50,000 pounds or 50 kips. Con-Serv's website also indicates that the allowable bearing capacity reduces with temperatures increasing above 70 degrees Fahrenheit, as shown in Figure 14. Con-Serv also provides the sliding friction coefficient as a function of load (Figure 15). The results of capacity checks of these slide bearings are provided later in this report, along with checks of structural members.

5.2.2. Coordination with Architectural Drawings

We also noted the following during our review of the architectural drawings.

Coordination. The structural layout of the building is in general conformance with the architectural drawings.

Mechanical Equipment. Spaces for mechanical equipment are shown on Sheets HQ-A2-1-2A, HQ-A2-1-3D, HQ-A2-1-04, and HQ-A2-2-2D. Sheets HQ-S0-2-01 through 02 from the structural drawings indicate that a live load of 150 psf (for "Mechanical") was used at all of the spaces for mechanical equipment identified on the architectural drawings except for that shown on HQ-A2-2-2D (at Level 2, bounded by Gridlines H, J, 14, and 16). Therefore, mechanical equipment indicated in the architectural drawings have generally been accommodated in the structural design drawings, except for the one aforementioned area.

5.3. Geotechnical Report Review

The geotechnical investigation at the site of the FLPHQ was conducted by Nutting Engineers of Florida, Inc. (Nutting), and their findings were summarized in a report dated January 20, 2021. Nutting characterized the soil at the project site as consisting of dense sand and soft rock. Accordingly, they designated the project site as Site Class D. Furthermore, the geotechnical report recommended implementing ground-improvement measures to enhance the bearing capacity of the soils. After vibro-replacement of the soil was performed, the report recommended that an allowable bearing capacity of 7,000 psf be used to design the structure. Sheet HQ-S0-1-02 of the structural drawings indicates that the design of the structure used soil Site Class D and an allowable bearing capacity of 7,000 psf as design parameters, consistent with the geotechnical report. No coefficient of friction between the soil and the foundations was set forth in the geotechnical report for use in the design of foundations for lateral loads. Additionally, a modulus of subgrade reaction was not reported.

5.4. Special Inspection Reports Review

In addition to being the SEOR, TT was also the Threshold Inspector (TI) for the FLPHQ. WJE reviewed threshold inspection reports provided to us (Reports No. 001 through 143, and 189 through 207) for our Phase 2 work. We reviewed these reports to understand how the interior masonry walls were constructed in the main portion of the FLPHQ, since the SEOR has claimed that they are using some of these walls as part of the lateral load resisting system (as will be discussed later). Below are excerpts from various inspection reports regarding CMU walls.

Report No. 065, Dated November 25, 2023: The TI notes a condition for Pour 2 of the roof slab. Up to this point, no conditions were noted for full height CMU walls except for the masonry shaft wall just to the east of Gridline J and to the north of Gridline 14. See Report No. 060.

Report No. 073, Dated December 8, 2023: The TI notes that "CMU wall dowels placed on SOG pour 1 were outside the wall layout therefore they were cut and new dowels were placed by drilling and epoxying at a 3" depth." Additionally, the TI notes that "EOR approved to grout Area A CMU wall on its full length." Figure 16 shows photographs from this inspection report.

Report No. 074, Dated December 11, 2023: The TI notes that "Duck tails used for horizontal attachemnt [sic]of CMU walls to concrete columns instead of following detail 8/HQ-S7-1-02. EOR approved for interior walls where there are no openings such that the portion of CMU attached to existing concrete exceeds the width of one CMU block. Duck tails are fixed with Ultracon 1/4"x1-3/4" hex washer head screws"

Report No. 080, Dated December 19, 2023: The TI notes that "CMU walls to be braced to the soffit beams using PTA 420 HS anchors as per approved sumittal [sic] #042200-13.0 instead of detail 6/HQ-S7-1-02."

Report No. 081, Dated December 19, 2023: Comments by the TI are in regard to the concrete used in slab-on-ground Pour 3. However, photographs of CMU walls in the first story are also shown in the report (Figure 17). These photographs show the overall CMU wall construction at one location in the first story and the approved PTA anchors at the top of the wall. The length of plate at the top of the PTA anchors is aligned parallel with the length of the CMU wall in contrast with the rendering shown on the manufacturer's website (Figure 10).

Report No. 082, Dated December 21, 2023: The TI notes that "CMU walls on the first floor are to be exposed and contractor does not want any cleanouts on the blocks. Therefore, walls from grid line 6 onward are to be poured in 4 ft lifts in order to properly tie the rebar for each lift."

Report No. 084, Dated January 2, 2024: The contents of this report are not in regard to CMU wall construction, but the included photographs (Figure 18) appear to show the first story with vertical reinforcement extending upward from the slab-on-ground, but prior to the CMU walls being fully installed. No shoring of the structure above is present, indicating that the CMU walls were not constructed as bearing walls.

Report No. 087, Dated January 8, 2024: The TI notes that "SOG pour 4 rebar. Dowels for CMU wall need to be rearranged [sic] so that the compy [sic] with detail 7/HQ-S7-1-02 for typical masonry wall on thickened slab." The TI also notes: "On CMU wall on lvl 1 area A grid line 4, rebar needs to be extended to the bottom of lvl 2 slab. See image on next page." See photographs from the report in Figure 19.

Report No. 088, Dated January 8, 2024: The TI notes regarding the issues noted in Report No. 087 above that "All the issues described on C001 of the previous report were verified to have been fixed prior to slab pour." and "Rebar on CMU wall along grid line 4 was extended to meet the bottom of the lvl 2 slab."

Report No. 097, Dated January 25, 2024: The TI notes that "Level 1 Area E mansonry [sic] wall grouted." No photographs of the wall were provided in the report.

Report No. 099, Dated January 29, 2024: The TI notes that (Figure 20) "HQ Level 3 South CMU wall, turn down vertical reinforcement that was added to slab pour was cut to place blocks and new vertical rebar was tied to the top slab using PTA 420 top anchors. See image for clarification." In Figure 20 the length of plate at the top of the PTA anchors is not consistently aligned either parallel or perpendicular with the length of the CMU wall. No shoring of the structure above is present, indicating that the CMU walls were not constructed as bearing walls.

Report No. 100, Dated January 30, 2024: The TI notes that "CMU wall on HQ Level 1 area E was poured using a self consolidating pearock 5000 psi concrete mix. EOR takes no exceptions but it is free to reject it."

Report No. 101, Dated January 31, 2024: The TI notes that "Checked vertical reinforcement of Level 3 area C CMU wall and there were two bars that were not tied to the coming dowels from previous block pour." Photographs were included that appear to be of the third floor slab (Figure 21), but no photographs were included for the referenced condition.

Report No. 103, Dated February 2, 2024: The TI notes that "Inspected CMU wall on Level 1 Area C. Some vertical bars needed to added before the next line of blocks is placed as to obtain proper lap splice length for next line of vertical reinforcement." Photographs (Figure 22) of these walls were included in the report. No shoring of the structure above is present, indicating that the CMU walls were not constructed as bearing walls.

Report No. 105, Dated February 5, 2024: The TI notes that they "Verified that horizontal ladder reinforcement from CMU wall extends more than 4 in into the C5 tie columns on CMU wall on Level 1 Area E." Photographs of this area are shown (Figure 23) indicating that the wall was constructed prior to the roof structure and therefore this wall would be a bearing wall.

Report No. 107, Dated February 8, 2024: The TI notes that "Partial inspection of slab rebar for SOG pour 5. Dowels for CMU need to be lowered to comply with what is specified on detail 7/HQ-S7-1-02." Figure 24 shows a photograph of this condition and a photograph of a CMU wall under construction. Again, note that no shoring of the structure above is present, indicating that the CMU wall was not constructed as a bearing wall.

Report No. 109, Dated February 12, 2024: CMU walls are not the subject of this report, but photographs in the report show reinforcing dowels extending upwards from the slab below (Figure 25). These dowels appear to be from the CMU wall below.

Report No. 123, Dated March 4, 2024: The TI notes that "Anchors to attach the vertical reinforcement of CMU walls to soffit beams were placed with one tapcon screw instead of two how it's specified on the product specs." Photographs of this condition are shown in Figure 26. The length of plate at the top of the PTA anchors is aligned parallel with the length of the CMU wall in contrast with the rendering shown on the manufacturer's website (Figure 10). No comments were made by the TI regarding the orientation of the anchor plate relative to the wall.

Report No. 125, Dated November 27, 2023 (Note that Report No. 124 is dated March 6, 2024 and Report No. 126 is dated March 12, 2024): The TI notes that "Confirmed that vertical reinforcement anchors are being tied as per specification using two tapcon screws instead of one." This statement appears to refer to the TI's comment in Report No. 123 above. A photograph of this condition was provided (Figure 27).

Report No. 131, Dated March 19, 2024: The TI notes that "Vertical reinforcement of Level 1 area D CMU checked for adequate lap splice." Two photographs are shown of a wall that appears nearly complete (Figure 28).

Report No. 133, Dated March 21, 2024: The TI notes that "Inspection of CMU on Level 1 area D between grid lines 15 and 16. Checked anchors are properly installed and rebar tied to them." See Figure 29.

5.5. Discussion of Design Basis

The magnitudes of the superimposed dead loads shown on Sheets HQ-S0-2-01 through 02 are reasonable for the type of finishes indicated on the architectural drawings, and the equipment (ductwork, piping, etc.) indicated on the mechanical, electrical, and plumbing (MEP) drawings.

The live loads shown on Sheets HQ-S0-2-01 through 02 for the elevated floors are appropriate based on the function of the space indicated on the architectural design drawings. No live load allowance is indicated for the slab-on ground, and it generally would not be necessary to indicate a live load allowance unless unusually large loads were expected on the slab.

Most of the areas with mechanical equipment identified in the architectural drawings were accommodated in the structural drawings in the loading diagrams. However, we have identified one area on Level 2, south of Gridline 14, that was designated in the architectural drawings as a mechanical room but was not designated as such on Sheets HQ-S0-2-01 through 02 (Figure 30). It should be confirmed whether mechanical equipment will be installed in this area and if so, if it has been accounted for in the structural design. For our structural analysis (discussed later), we have assumed that the loading diagrams provided in the structural drawings are correct. Additionally, in some locations on the roof, both the weight of the mechanical equipment and the 150 psf allowance in live load are shown. We recommend that the SEOR confirms which load or combination of loads governed the design of the structure and specify so on the drawings. Currently, where equipment is installed, we do not know whether the equipment weight, or the specified 150 psf live load, or a combination of the two was used by the SEOR to design the structure supporting the equipment, Moreover, we recommend that the SEOR confirms if there is mechanical equipment in the area bounded by Gridlines H, J, 14, and 16 (or other areas which currently are not reflected in the structural drawings).

We also recommend that the structural drawings be updated to address the inconsistency in column locations shown on the plan views and column schedule. It is likely that the columns indicated on "Gridline K.8" in the column schedule actually refer to those on Gridline K.6 on the plan views.

No information is provided on the permitted structural drawings about how rain load was considered in the structural design. It is likely that the uniform roof live loads are in excess of the code-required rain loads, but we recommend that the SEOR indicate on the structural drawings the specific rain loads that were used in the design or confirm that they do not control the design of the FLPHQ. Moreover, no information related to design for flood loads was found in the structural drawings. The site of the FLPHQ is in Zone X in the 2014 FEMA flood maps, which would have been used for the design of FLPHQ. However, the 2024 FEMA Flood Maps indicate that the site of the FLPHQ is in Zone AE. Based on the flood maps in effect at the time of the design we understand that design for flood loads was not required (since Zone X of the 2014 FEMA flood maps is exempted from flood design requirements).

The wind load design criteria shown in the structural drawings are appropriate for this structure, and we concur with the SEOR that the FLPHQ is a Risk Category IV structure. Furthermore, the seismic design criteria are appropriate if the lateral-load-resisting system consists of ordinary reinforced concrete shear walls as the permitted structural drawings indicate. However, the seismic design criteria are not appropriate if the system includes ordinary reinforced masonry shear walls, as the SEOR has indicated in their lateral-load system calculations package submitted to WJE on March 24, 2025. In the event that the SEOR is including reinforced masonry shear walls as part of the lateral system, the R value of 4 that the SEOR used in the design would need to be changed to 2, meaning the seismic design force would double.

As will be discussed in greater detail later in this report, the special inspection reports generally support our conclusion that the masonry partition walls were not an original design component of the lateral load-resisting system. The staging of wall construction and connection of the walls to the above floors above (in the main portion of the FLPHQ, i.e., not Area E) are not consistent with reasonable practice for designing and constructing reinforced masonry shear walls. Another issue may be related to the orientation used for installation of the PTA anchors, which some of the special inspection reports highlight (e.g., see Figure 17 and Figure 26) as not being orthogonal to the CMU walls as is shown on the H&B website (Figure 10). We recommend that the SEOR comments on which orientation of the anchors is appropriate. Furthermore, we recommend that the SEOR provides a) documentation from the anchor manufacturer which supports the use of that orientation, and b) a verifiable load resistance value associated with that orientation. Without this information, it is not clear to us how the CMU partition walls could be fairly utilized as shear walls.

The soil coefficient of friction is not provided on the structural drawings but is used in the lateral-load system calculations package. At the same time, if the lateral system is envisioned as employing the CMU walls to resist wind and seismic forces, a verifiable coefficient of friction for construction on a vapor retarder should also be obtained. We recommend that the SEOR includes these values on the structural drawings.

6. REVIEW OF THE STRUCTURAL DESIGN

In addition to reviewing the design basis for the FLPHQ, we performed a more detailed review of the structural system by checking limited members and conditions for code-compliance, considering strength, structural integrity, detailing, serviceability, and durability. Note that we evaluated both Area E (the community room and lobby area) and the main portion of the building, east of Gridline 2. We also reviewed a calculations package developed by the SEOR concerning their assumptions regarding the lateral load-resisting system of the FLPHQ.

6.1. Development of Numerical Model

To facilitate our review, we developed structural analysis models of the FLPHQ in the software program, ETABS, by Computers and Structures, Inc. (CSI), which is commonly used in the design and analysis of buildings. After applying the design loads to the model, we extracted the internal forces (design demands) and deflections of members (among other parameters of interest) to compare with the design capacity of structural members (beams, columns, walls, etc.).

The geometry, loading, and material properties used in our model were taken from the structural drawings. To model the increase in flexibility in reinforced concrete elements upon cracking (which is typical at factored load levels), we applied stiffness modifiers to the reinforced concrete slabs, joists, beams, columns, and walls comprising the structural system. The stiffness modifiers were selected based on the ACI 318-14 building code (which was applicable for the design of the FLPHQ) and recommendations from CSI (the developers of ETABS). The enlargements and other modifications to the superstructure that occurred after deflection of the cantilevered roof beams during construction were incorporated into our model. As will be described later in this report, we do not believe that the partition masonry walls are a reliable feature of the lateral load-resisting system. Therefore, we did not include them in our numerical model. However, we did include the reinforced concrete shear walls, slabs and frames.

The main portion of the FLPHQ was contained in one model, while Area E was modeled as a separate building, due to the expansion joint between these two portions of the building. Area E has lateral load-resisting systems that are different in two orthogonal directions. In its east-west direction, a CMU bearing wall with concrete tie column boundary elements is used to resist load. Note that, unlike the partition masonry walls in the main portion of the FLPHQ, this bearing wall was detailed and constructed to function as a masonry shear wall (as indicated in the drawings and in the special inspection reports); therefore, we did include it in our numerical model of Area E. In its north-south direction, the lateral load is resisted by a reinforced concrete moment-resisting frame. When evaluating the structure for Area E, our numerical model considered these two systems. Since Area E relies on a moment-resisting frame for lateral support, we considered several boundary conditions at the base of the columns. We considered "pinned" column base supports that preclude translation, but allow rotation, "fixed" base supports that preclude translation and rotation, and "stiff" base supports that consider some of the flexibility of the footing and the soil.

6.2. Structural Member Strength Capacity Checks

We performed strength capacity checks on a representative fraction of structural elements. As the scope of our work was primarily a structural peer review, we did not attempt to check every condition and element in the building, but rather used our experience and judgement to evaluate typical and potentially critical conditions. The strength limit states that we evaluated for each type of structural element within the structure (including Area E) are summarized below.

Slab Limit States: One-way flexure, one-way shear, and transfer of diaphragm shear into the shear walls (for the portion of structure bounded by Gridlines 2, 16, F, and L)

Joist Limit States: One-way flexure and one-way shear

Beam Limit States: One-way flexure, one-way shear, and combined shear and torsion

Column Limit States: Combined axial-flexural loading, one-way shear, and bearing

Wall Limit States: Combined axial-flexural loading and one-way shear

Foundation Limit States: Soil bearing, overturning, sliding, one-way flexure, one-way shear, and punching (two-way) shear

Slide Bearing Limit States. Bearing

The extent to which a member satisfied code-required strength levels was quantified using the ratio of design demand to design capacity, (i.e., demand-to-capacity ratio or DCR). A DCR greater than 1.0, i.e., unity, indicates that the demand is greater than the capacity, meaning the element is overstressed, while a DCR less than 1.0 indicates that the design demands are less than the design capacity.

Structural members are required by code to have a margin of safety against design demands. Therefore, a DCR greater than 1.0 does not necessarily mean that the member will fail, but rather that the member does not meet the prescribed margin of safety required by the applicable code. The margin of safety required by code is usually achieved though strength adjustment factors when calculating the capacity of the member or a combination of strength adjustment factors and factored (increased) load demands. The calculation of the capacity of a structural member for most loading types is also usually carried out using specified strengths of the materials used in the construction.

6.3. Review of Lateral Load Calculations Package from the SEOR

The SEOR provided a calculation package (dated March 21, 2025) that describes their assumptions regarding the design of the lateral load-resisting system of the FLPHQ. However, a number of these assumptions appear to be inconsistent with the original design as set forth in the structural drawings. It is unclear to us if the inconsistencies that we have identified derive from errors made when the design was being developed, errors made when this calculation package was being developed (almost three years after issuance of the 100% construction drawings) or from a series of misunderstandings. These inconsistencies (in concert with the special inspection reports), discussed in Section 6.6.1 of this report, lead us to believe that the masonry walls described in the calculations package were never incorporated into the original design of the lateral system for the FLPHQ. Note that WJE did not receive original calculations for the lateral system which predated the construction of the FLPHQ and request that those calculations are shared with us for review. For this report, we simply set forth below basic information from the March 21, 2025, calculations package.

On Sheet 2 of the document, the SEOR claimed that "The lateral system for the Fort Lauderdale Police Headquarters was designed utilizing a combination of cast-in-place (CIP) concrete shear walls and concrete masonry unit (CMU) shear walls." Sheet 2 also provided assumptions regarding stiffness modifiers for the SEOR's numerical model which affect the distribution of forces in the structure. The SEOR also noted the stiffness modifiers that they claimed to have used (summarized below).

- Stiffness modifiers of 1.057 and 1.078 were used for the 8-inch and 12-inch thick reinforced concrete shear walls. The SEOR claimed that the walls were uncracked and used transformed section analysis to justify using a stiffness modifier greater than 1.00.
- The slabs and joists were also noted as being uncracked, indicating that stiffness modifiers of 1.00 were used for both.
- Stiffness modifiers ranging between 0.25 and 0.50 were used for the CMU shear walls on the basis that the walls were considered cracked.
- Stiffness modifiers of 0.35 and 0.70 were used for the soffit beams and columns, respectively.

Sheets 8 through 11 contained the output from the SEOR's numerical model and accompanying flexural and shear capacity checks for the reinforced concrete shear walls. The output indicates that the flexural and shear DCRs of wall segments ranged from approximately 0.01 to 1.02 and 0.60 to 0.71, respectively.

There is but one CMU core wall which extends over the full height of the building (bounded by Gridlines J, K, 13, and 14) and has its own unique foundation system embedded 5 feet below grade, thereby allowing it to resist lateral translation via a combination of passive pressure and friction. However, the remaining CMU walls on the Ground Level did not have their own footings. As such, these walls rely solely on friction under the slab on ground to transfer lateral load from the walls to the ground, despite that a vapor retarder is understood to be present between the slab on ground and soil beneath. The SEOR has claimed that the geotechnical engineer (Nutting) approved of a coefficient of friction of 0.3 when determining the sliding resistance under the slab on ground that supports the typical CMU walls, which is the same coefficient used to determine the sliding resistance of the footings (which do not have vapor retarders under them). Note that WJE was not provided documentation by Nutting approving this friction coefficient value, without which an obvious question exists regarding how the two different conditions warrant the same friction coefficient.

The SEOR summed the following sources of axial load when calculating the normal force below the CMU walls which allows for the development of frictional resistance against sliding of the wall:

> The axial load in the wall due to loads imposed from “postulated” tributary construction from the remainder of the building. Note that we characterize this as “postulated” to highlight that because the CMU walls were actually constructed after the concrete framing of the building, there is little-to-no “actual” tributary gravity loading on these walls, meaning their assumption that there is tributary loading is unconservative and erroneous;
>
> The self-weight of the CMU walls;
>
> The self-weight of the slab on ground in an area surrounding the wall, including thickened portions; and
>
> The weight of the holding-cell walls, roof, and exterior wall footings, as applicable.

The calculations represent that the SEOR used axial and shear loads for allowable stress design (ASD) load combinations from their ETABS model (which appears to include both the walls and frames for the lateral analysis), although they did not report what those loads were. However, they did report the loads in the CMU walls for load and resistance factor design (LRFD) load combinations. The frictional resistance and lateral demands on those walls were shown on Sheet 61 of the calculations package. A square footage of slab on ground which the SEOR utilized in computing the frictional resistance was shown on the same sheet. The minimum, average and maximum ratios of demand-to-available frictional resistance for the 13 walls (or groups of walls) shown on that sheet were approximately 0.90, 0.98, and 1.00. The SEOR did not provide an example calculation showing how the frictional resistance was calculated. The lateral load calculations package also did not address transfer of shear forces from the diaphragms into the CMU walls. Regardless, the combination of a) the SEOR's reliance on postulated and unconservative tributary gravity loads, and b) the reported DCRs being close to or at 1.0 suggest that the actual available frictional resistance under these walls is inadequate.

6.4. Structural Integrity, Reinforcement, Serviceability, and Durability Requirements

In addition to capacity checks, we also reviewed portions of the structure for compliance with structural-integrity, reinforcement, serviceability, and durability requirements of applicable codes.

6.4.1. Structural Integrity Requirements

We reviewed the structural design for compliance with the following structural-integrity requirements from the FBC and ACI 318-14. A brief summary of each requirement follows the provision number.

FBC 1615.3.1: Concrete frame structures must satisfy the provisions of Section 4.10 of ACI 318. Where ACI 318 requires that reinforcement passes through column cores, that reinforcement must have a minimum tensile strength of two-thirds the required one-way vertical shear strength of the floor-to-column connection. The tensile strength may be as low as one-third the required shear strength when a monolithic/bonded slab is present with a minimum ratio of continuous reinforcement of 0.0015 times the concrete area in either direction.

ACI 318.9.7.7.1(a): At least one-quarter of the maximum positive moment reinforcement, but not less than two bars or strands, shall be continuous in perimeter beams.

ACI 318.9.7.7.1(b): At least one-sixth of the maximum negative moment reinforcement, but not less than two bars or strands, shall be continuous in perimeter beams.

ACI 318.9.7.7.1(c): Longitudinal structural integrity reinforcement in perimeter beams shall be enclosed by closed stirrups in accordance with 25.7.1.6 or hoops along the clear span of the beam.

ACI 318.9.7.7.2(a): At least one-quarter of the maximum positive moment reinforcement, but not less than two bars or strands, shall be continuous in beams not on the perimeter of the structure.

ACI 318.9.7.7.1(b): Longitudinal structural integrity reinforcement in beams not on the perimeter of the structure shall be enclosed by closed stirrups in accordance with 25.7.1.6 or hoops along the clear span of the beam.

ACI 318.9.7.7.3: Longitudinal structural integrity reinforcement shall pass through the region bounded by the longitudinal reinforcement of the column.

ACI 318.9.7.7.4: Longitudinal structural integrity reinforcement shall be anchored to develop the reinforcement yield stress at the face of the support

ACI 318.9.7.7.5(a): Positive moment reinforcement shall be spliced at or near the support.

ACI 318.9.7.7.5(b): Negative moment reinforcement shall be spliced at or near midspan.

ACI 318 9.7.7.6: Splices shall be full mechanical, full welded, or Class B tension lap splices.

6.4.2. Reinforcement Limit and Detailing Requirements

We reviewed the structural design for compliance with reinforcement limit and detailing requirements of ACI 318-14. We checked the lap splice and development length schedules from Sheet HQ-S4-5-01 to understand if those lengths were equal to or exceeded the minimum lengths required by ACI 318-14. ACI 318-14 Section 10.6.2.1 also requires the use of a minimum amount (and maximum spacing) of transverse reinforcement in beams and columns to resist one-way shear forces when the factored demand exceeds one half of the code-calculated shear capacity of the concrete.

6.4.3. Serviceability and Durability Requirements

We also reviewed the structural design for compliance with serviceability and durability requirements from ACI 318. Specifically, we checked that a) the cover depths met or exceeded ACI requirements, b) a minimum amount of slab crack-control reinforcement was provided, and c) beam deflection limits were satisfied. We also checked the service-level story drifts of the FLPHQ against the recommended limit of 1/600 to 1/400 of the story height, provided in the commentary to Appendix C of ASCE/SEI 7-16. These common drift limits are intended to minimize serviceability issues (e.g., damage to cladding and nonstructural walls) that often stem from inter-story drift caused by wind loads. We also reviewed the modal response of the building (using our ETABS model) to understand if there existed an unusual dynamic response of the FLPHQ, relative to similar three-story structures.

6.5. Findings

Findings from our review of the structural system related to capacity, structural integrity, reinforcement, serviceability, and durability are provided below.

6.5.1. Capacity Checks

The results of the capacity checks we performed for the elements south of Gridline 2 are organized in accordance with each type of structural element. See the Discussion section below for further information about conditions which did not satisfy code requirements for structural capacity.

Slabs. For the elevated slabs, the DCRs for flexure, one-way shear, and transfer of diaphragm shear into the shear walls were below unity, meaning the slabs were in compliance with respect to the applicable strength requirements.

Joists. The DCRs for flexure and one-way shear for the joists were below unity, meaning the slabs were in compliance with respect to the applicable strength requirements.

Beams. The DCRs in flexure, one-way shear, or combined shear and torsion for several beams near the mechanical equipment on the roof (e.g., RSB-15) exceeded unity. Otherwise, the beam DCRs were below unity.

Columns. The DCRs for combined axial and flexural loading exceeded unity for 24 columns, meaning those columns were not in compliance with respect to the applicable strength requirements (Table 1). Of those columns, 7 were overstressed (i.e, the DCR was greater than unity) by 5% or less, 9 were overstressed between 5 and 10%, and 8 were overstressed by more than 10%. Moreover, 23 columns (identified in Table 2) were overstressed for one-way shear. Of those columns, 3 were overstressed (i.e, the DCR was greater than unity) by 5% or less, 2 were overstressed between 5 and 10%, and 18 were overstressed by more than 10%. If the transverse reinforcement spaced more widely than one-half of the effective depth was used to determine capacity following the provisions of ACI 369.1-22 (an alternative code not typically applied to new buildings), as will be discussed later, the number of overstressed columns reduced to 0. More to the point, 50 columns were sufficiently stressed in shear such that, ACI 318-14, the concrete code governing the design, required that a minimum amount of transverse reinforcement be provided that was spaced no more widely than one-half of the effective depth. However, the spacing specified in the structural drawings exceeded one-half of the effective depth. As such, the transverse reinforcement in these 50 columns is not in compliance with the relevant code requirements. Those columns are identified in Table 3. For 48 of those columns, the shear demand exceeded one-half of the reduced nominal concrete shear strength (the limit beyond which minimum reinforcement must be provided) by more than 10%. Compliance with this prescriptive code minimum transverse reinforcement requirement would have resolved the previously noted one-way shear overstresses.

Walls. The reinforced concrete shear walls were code-compliant for axial-flexural loading, except for the first story of the south elevator shaft, meaning that this portion of the wall is not in compliance with the relevant code requirements. The factored shear force and flexural demand in that portion of the wall exceeded the design capacities for shear and flexure (calculated in accordance with ACI 318-14).

Foundations. Some foundations are not in compliance with the relevant code requirements in several respects. The DCR for soil bearing stress exceeded the soil bearing capacity (7,000 pounds per square foot) under the three isolated footings on Gridline 4, between Gridlines G and K. The sliding shear force due to east-west winds exceeded the allowable resistance for mat footings CF-1 and CF-3 (under the northern-most stairwell and the southern-most elevator, respectively), and the overturning moment due to east-west winds exceeded the allowable restoring moment for mat footing CF-3 as well.

Slide Bearing Limit States. The service-level stress in the slide bearings was found to exceed the allowable bearing stress reported by the bearing manufacturer. The axial load demand in the bearing exceeded 50 kips divided over the 25 square inch bearing area.

Table 1. Locations of columns with axial-flexural demand-to-capacity ratios (DCRs) exceeding unity

Column ID	Story	Column ID	Story	Column ID	Story
G4[c]	3	G11[b]	3	K8[b]	3
G5[a]	3	G16[a]	3	K9[c]	3
G6[b]	3	H9[c]	2	K10[c]	3
G7[b]	3	H10[c]	2	K11[c]	3
G8[b]	3	H16[a]	3	K12[c]	3
G9[b]	2	J9[a]	2	K13[a]	3
G9[b]	3	J9[a]	3	K15[b]	3
G10[b]	3	K6[a]	3	K16[c]	3

Notes: Column ID is defined by the intersection of perpendicular gridlines

[a]DCR does not exceed 1.05; [b]DCR is between 1.05 and 1.10; [c]DCR exceeds 1.10

Table 2. Locations of columns with one-way shear demand-to-capacity ratios (DCRs) exceeding unity

Column ID	Story	Column ID	Story	Column ID	Story
G3[c]	3	G11[c]	3	K9[c]	3
G4[c]	3	G12[a]	3	K10[c]	3
G5[c]	3	G13[b]	3	K11[c]	3
G6[c]	3	G14[a]	3	K12[c]	3

Column ID	Story	Column ID	Story	Column ID	Story
G7[c]	3	K5[a]	3	K13[c]	3
G8[c]	3	K6[c]	3	K15[c]	3
G9[c]	3	K7[b]	3	K16[c]	3
G10[c]	3	K8[c]	3		

Notes: Column ID is defined by the intersection of perpendicular gridlines

[a]DCR does not exceed 1.05; [b]DCR is between 1.05 and 1.10; [c]DCR exceeds 1.10

Table 3. Locations of columns which do not have minimum transverse reinforcement that is spaced less widely than the maximum spacing prescribed by ACI 318-14

Column ID	Story	Column ID	Story	Column ID	Story
G3	3	G13	3	J10	2
G4	2	G14	3	J10	3
G4	3	G15	3	K3	3
G5	3	G16	3	K4	2
G6	2	H8	2	K5	3
G6	3	H8	3	K6	3
G7	2	H9	2	K7	3
G7	3	H9	3	K8	3
G8	2	H10	2	K9	3
G8	3	H10	3	K10	3
G9	2	H16	2	K11	3
G9	3	H16	3	K12	3
G10	2	J5	2	K13	3
G10	3	J5	3	K14	3
G11	2	J8	3	K15	3
G11	3	J9	2	K16	3
G12	3	J9	3		

Notes: Column ID is defined by the intersection of perpendicular gridlines. The factored shear force exceeded the reduced nominal concrete shear strength by more than 10% for all columns except two: Columns G4 and H8.

6.5.2. Structural Integrity, Reinforcement, Serviceability, and Durability

The other findings from our review of the structural design for conditions not directly related to member capacity are provided below.

Structural Integrity. The structural integrity provisions of ACI 318 and the FBC were generally adhered to in the design of the FLPHQ. However, it appears that noncontact lap splices of prestressing strand to "shear-friction bars" were relied on to establish continuity of bottom reinforcement in the soffit beams. This type of detail is not addressed in either ACI 318 or the FBC; it is therefore not clear how the SEOR is justifying this aspect of the design.

Reinforcement Limits and Detailing. The provided slab flexural reinforcement satisfied the minimum requirements from Section 7.6.1.1 of ACI 318-14. The calculated lap-splice and development lengths from Sheet HQ-S4-5-01 of the structural drawings meet or exceed the minimum lengths from ACI 318-14.

Serviceability. We noted the following serviceability-related conditions.

- When the main building was subjected to service-level lateral load, the story drifts were less than 1/600 of the story heights. Additionally, the beam deflections under gravity load were less than the maximum limits prescribed in ACI 318-14. The first period of vibration of the structure was approximately 0.94 seconds and included a significant twisting or torsional mode of behavior. A similar displaced shape of the structure was observed to occur when subjected to east-west wind loading.
- When Area E was subjected to service-level lateral load, we found that this portion of the FLPHQ was very stiff in the east-west direction. However, it was much more flexible in the north-south direction. The first period of vibration of the structure ranged from approximately 0.86 seconds (with fixed base columns assumed) to 1.9 seconds (with pinned base columns assumed) and also included a significant twisting mode of behavior. The geometry of Area E with the radiused walls and sloped roof is such that the lateral deflection of the roof, and thus the drift, can vary significantly based on the location. Under south-to-north wind loads, the greatest lateral deflection occurs at the column at Grid Point A/A.1 while the columns near Grid Point F/3 have the least lateral deflection. The greatest lateral deflection may be more than 10 times greater than the least lateral deflection. The assumption regarding column base fixity also significantly influences the lateral deflection at the top of the columns. Changing the columns from fixed base to pinned base can increase the lateral deflection at the roof under south-to-north wind by a factor of more than 3 times. Even with fixed-base columns, some columns can exceed a drift ratio of 1/400 of the column height.

Durability. The cover depths shown in the structural drawings satisfy code-prescribed minimum values. However, the dimension lines shown in the precast concrete shop drawings (e.g., Sheet SC-3C-D of Submittal 328) were inconsistent and implied different cover depths. Figure 31 and Figure 32 illustrate one example of this inconsistency. For the first detail (Figure 31), the 1.5-inch dimension was drawn from the top of slab to the transverse reinforcement. For the second detail (Figure 32), it was drawn from the top of slab to the center of the top longitudinal reinforcement such that there was approximately zero inches of cover to the transverse reinforcement. We are unable to ascertain if what was constructed conforms to the cover requirements set forth in Figure 31 or Figure 32.

6.6. Discussion

Conceptually, the FLPHQ has complete gravity- and lateral-load paths. Within the gravity load-resisting system, loads are applied to or result from the reinforced concrete slabs, which transfer those loads to precast concrete joists. Load from the joists is resisted by the soffit-beam system, which is supported by the cast-in-place concrete columns and foundations. For the lateral load-resisting system, wind (as the controlling lateral load) is applied to the building envelope, which transfers the load to the reinforced concrete slabs, which act as diaphragms. Lateral load in the diaphragms is transferred to the reinforced concrete shear walls through interface shear. Load in the walls is transferred into the ground through a combination of friction under the footing, passive earth pressure in front of the footing, and soil bearing.

The following sections provide further discussion regarding our findings concerning the lateral load-resisting system of the FLPHQ and deficiencies in the design of the structure that we identified during our peer review. We recommend that the SEOR consider carefully these findings and make the necessary rectifications to the structure.

6.6.1. Use of Partition Masonry Walls as Part of the Lateral Load-Resisting System

While we do acknowledge that masonry walls, and even infill masonry walls, can resist lateral forces on buildings if properly designed and detailed, based on our review of the lateral system calculation package prepared by the SEOR (dated March 21, 2025), we do not agree that the partition masonry walls should be considered as part of the lateral load-resisting system for the main portion the FLPHQ. Moreover, based on the information provided in the permit drawings for the project, we are not convinced that the SEOR considered the masonry walls to be part of the lateral system at the time of design. Regardless, we have a number of concerns with the SEOR's assumptions and design that are presented in the calculations package recently provided to us. Those concerns preclude us from agreeing that the walls are a reliable part of the lateral load-resisting system to the degree that the SEOR is attempting to utilize them.

Concrete Shear Wall Stiffness Modifier. We do not agree with the stiffness modifier which the SEOR has used for the concrete shear walls in their numerical model. The SEOR used stiffness modifiers greater than 1.0 for the cast-in-place concrete shear walls on the basis that they believed the walls to be uncracked. It is generally implausible for a reinforced concrete element to be uncracked when subjected to factored design loads, since reinforced concrete develops its flexural strength by the straining of reinforcement after cracking. While axial loading may delay the onset of cracking that stems from external loading, the concrete shear walls have a small gravity tributary area and are only lightly axially loaded. Therefore, it is highly unlikely (as our analysis indicates) that the walls are uncracked at factored load levels. The SEOR's calculations indicate that the DCRs in flexure and shear of the walls are as high as 1.02 and 0.71, respectively. Considering that a) reinforced concrete cracks under load that is often low relative to its ultimate capacity, b) the SEOR's calculations indicate that portions of the walls are highly loaded, and c) restraint stresses from (e.g.,) drying shrinkage in the walls promote cracking even in the absence of external load, it is implausible for the walls to be uncracked at factored loading. Cracking in the walls would require the use of a stiffness modifier less than 1.0 in the analysis which would cause more load to be redistributed to the masonry walls (to the extent that they reliably participate in the lateral load-resisting system). Even if the walls were uncracked, no stiffness modifier should be greater than 1.0. Both considering the concrete shear walls as uncracked and using a stiffness modifier greater than 1.0 manner appear to represent an attempt to tune the model artificially to allow the CMU walls to take some load, but not "too much." The SEOR's calculations package indicates that the frictional resistance against sliding of all CMU walls is nearly equal to their accompanying demands—even with a stiffness modifier greater than 1.0, used for the concrete shear walls. Therefore, the additional load expected to be added to the CMU walls after using an appropriate stiffness modifier would likely cause the total wall demands to exceed the frictional resistance against sliding.

Concrete Column Stiffness Modifier. Due to their flexural stiffness, the reinforced concrete columns will also resist some shear that would otherwise be distributed to the shear walls. We agree with the SEOR that a flexural stiffness modifier for the columns of 0.70 (as used in the lateral system calculations package) is appropriate for the building (as supported by ACI 318-14 Table 6.6.3.1.1(a)). However, this modifier is not consistent with other values that the SEOR has indicated are appropriate in previous documents which we have received. As described in Section 4.2 of this report, the SEOR contended that Columns G/2 and K/2 exhibited "pin-ended column behavior" (i.e., had an effective flexural stiffness modifier of 0.00) when positioning their argument as to why these columns did not need to satisfy the aforementioned ACI 318-14 prescriptive requirement for minimum transverse reinforcement spaced no more widely than one-half the effective depth. They used this to contend that the shear in the columns triggering the ACI 318-14 requirement for minimum transverse reinforcement did not exist. Furthermore, the SEOR used a flexural stiffness modifier of 0.35 for Columns H/2 and J/2, resulting in a sufficiently small flexural demand in the columns and allowing them to use the larger of the two shear capacities permitted in Table 22.5.6.1 of ACI 318-14. Based on this stiffness modifier, they argued that these two columns also did not trigger the previously mentioned prescriptive code requirement for minimum transverse reinforcement. Neither assumption regarding column stiffness (i.e., flexural stiffness modifier) is consistent with the value of 0.70 that the SEOR used for assessing the lateral system in the FLPHQ. The higher column stiffness modifier used for the columns when assessing the lateral system reduces the amount of lateral demand resisted by the concrete shear walls and partition masonry walls, decreasing the DCRs in the walls. Therefore, while the values for the column stiffness modifier used by the SEOR are internally inconsistent across the documents which they have provided, they all consistently align with an objective of demonstrating that structural members are adequate. In other words, the SEOR appears to be using modeling assumptions, regardless of whether or not they are legitimate, consistent, or reasonably supported by code provisions, which are most favorable to their position. We recommend that the SEOR clarifies what stiffness modifier was used for the columns to design the FLPHQ, since that modifier will affect the distribution of forces within the structure.

Slab Stiffness Modifier. We also do not agree that the slab stiffness modifier that the SEOR used in their numerical model properly reflects the behavior of reinforced concrete slabs. The reinforced concrete slabs are not uncracked at factored loading, for similar reasons as described previously for the concrete shear walls. It is further evident that the slab would not be uncracked at factored loading by the fact that cracks have already been observed and repaired near Gridline 2 (Figure 33). Since the distribution of forces in the FLPHQ is contingent on the stiffness of the diaphragm, the use of more appropriate stiffness modifiers may change the demands in the CMU walls. Given that the demands in those walls are already close to their design capacity (based on the SEOR's calculations), it is not evident that the walls will have sufficient capacity after the modifiers are updated.

Axial Load in the CMU Partition Walls. We do not agree that the CMU partition walls carry substantial axial load beyond their own self-weight. Due to the construction sequence of the FLPHQ (evidenced in the special inspection reports), the CMU partition walls were "infill" that was installed long after the structural framing (columns, beams, joists, slabs) was constructed. Apart from potentially some superimposed dead load installed after constructing the partition walls and possibly live load, we do not expect substantial axial load to be resisted by the partition walls beyond their self-weight. This is especially the case because Detail 6 of Sheet HQ-S7-1-02 of the structural drawings explicitly calls for a minimum one-inch gap between the bottom of the slab and the top of the CMU. We presume this is specified for the purpose of structurally isolating the walls from the floor diaphragm—to prevent transfer of both gravity and lateral load into the CMU walls. It is therefore highly confounding that Sheet 52 of the SEOR's calculation package indicates that the design of these so-called "CMU shear walls" is reliant on a degree of frictional resistance that can only be developed if the CMU walls support substantial gravity load beyond self-weight. For example, per the calculation package, CMU Wall 029 resists roughly 80 kips (1 kip = 1,000 pounds) of dead load as compared to the approximate self-weight of the wall of around 9 to 12 kips (which we calculated) depending on the number of grouted cells. Given that the SEOR's calculations indicate that the lateral demand on the walls nearly exceeds the sliding frictional resistance (even when the walls are wrongly assumed to support substantial axial load beyond their self-weight), and frictional resistance is directly proportional to the normal force (caused, in part, by axial force in the wall), it is doubtful that the walls would have sufficient resistance for sliding under lateral load after the axial load in the wall is corrected in their analysis.

Coefficient of Friction. The SEOR's lateral-load package generally relies on the use of a coefficient of friction of 0.3 under the slab on ground to develop frictional resistance. While this may be a reasonable design value for the mat foundations, it is not evident that this friction coefficient is appropriate for the slab on ground under the CMU walls. The structural drawings (e.g., Detail 7 from Sheet HQ-S7-1-02, shown in Figure 4) indicate that a vapor retarder was placed between the bottom of the slab on ground and the soil which may reduce the friction between the slab and soil. We were not provided the documentation from Nutting to support the claim that a friction coefficient of 0.3 is appropriate. Accordingly, we recommend that the SEOR provide documentation from Nutting in which they specifically confirmed that a value of 0.3 is reasonable when a vapor retarder is in place.

Diaphragm Shear Transfer. The lateral load calculations package does not address whether the connections between the concrete floor diaphragm and masonry walls are adequate to transfer shear from the diaphragm. Detail 6 of Sheet HQ-S7-1-02 (Figure 7) of the structural drawings indicate two details for connecting the masonry walls to the slab or soffit beams. The first option (the "typical detail") entails embedding the vertical wall reinforcement several inches into the diaphragm. Lateral load from the diaphragm might thereby be presumed to be transferred into the CMU walls through shear friction. However, Section 22.9.5.1 of ACI 318-14 requires that shear-friction reinforcement be developed on both sides of the friction interface, which the reinforcement in the wall is not, since per Section 25.4.2.1, the bars must be embedded at least 12 inches to develop the bar, which they are not. Therefore, this detail does not comply with the ACI 318-14 code if its intent is to transfer diaphragm shear through shear friction. The second option shown in Detail 6 (the "alt. option") entails anchoring two angles into the diaphragm on either side of the wall, presumably to laterally brace it. Since the angles are not anchored into the wall in the detail, lateral load from the diaphragm cannot be transferred into the walls. Moreover, a gap with a minimum thickness of 1 inch is shown between the top of wall and bottom of diaphragm, the intent of which appears to have been to preclude the wall from receiving diaphragm shear and floor gravity loading until the gap closes. Additionally, various threshold inspection reports (e.g., No. 123) show PTA 420 HS anchors (produced by Hohmann & Barnard, Inc.) which are post-installed into the soffit beams and slabs using Tapcon screws and tied to the vertical reinforcement from the CMU walls (Figure 17, Figure 20, Figure 26, and Figure 27). This connection cannot reasonably transfer diaphragm shear into the walls either. Threshold inspection report No. 80 notes "CMU walls to be braced to the soffit beams using PTA 420 HS anchors as per approved sumittal [sic] #0422000-13.0 instead of detail 6/HQ-S7-1-02," suggesting that these anchors were used for most connections of CMU walls to soffit beams. For these reasons, we recommend that the SEOR provide calculations for review demonstrating that the wall-to-diaphragm interface connections are adequate to transfer shear from the slabs into the walls.

While the SEOR has claimed in their lateral system calculations package that the CMU partitions were designed as shear walls, their structural drawings contradict this statement in several locations as noted above and below, meaning that the SEOR's current position of the CMU walls as an intended part of the lateral system is suspect.

1. The seismic response modification (R) factor shown on Sheet HQ-S0-1-01 of the structural drawings is 4, which is consistent with Table 12.2-1 of ASCE/SEI 7-16 for an ordinary reinforced concrete shear wall system but not for a system with CMU shearwalls. The R factor for an ordinary reinforced masonry shear wall system—which applies if the CMU partition walls are relied on as shear walls even in a system that also has reinforced concrete shearwalls —is 2. An R factor of 4 also would not be applicable to a hybrid system with both reinforced concrete and CMU walls because Section 12.2.3 of ASCE/SEI 7-16 requires that the lower R factor (2) be used for design.
2. The same sheet of the structural drawings describes the lateral system as "Ordinary Reinforced Concrete Shear Walls" and makes no mention of the CMU walls, again underscoring the specifics of the original design.
3. On numerous sheets in the structural drawings (e.g., Sheet HQ-S2-2-1A), the CMU partition walls are referred to as "non-loadbearing CMU", which is inconsistent with the SEOR's claim in their lateral system calculation package that the partition walls carry axial load. This is also inconsistent with the SEOR's current position that frictional resistance at the base of the walls is an important aspect of the lateral resistance of the building.
4. The structural plan notes indicate that for additional information on the "lateral system elevations, connection forces and details" to refer to the "S3 Series Drawings." Sheets HQ-S3-1-01, HQ-S3-1-03, HQ-S3-1-04 and HQ-S3-1-05 (the S3 series, note HQ-S3-1-02 either does not exist or was not shared with WJE) pertain only to reinforced concrete walls, again demonstrating that the SEOR has only lately posited that the masonry walls are part of the lateral system. If the masonry walls are

being treated as shear walls, they should be clearly designated on these sheets (and other locations as appropriate) to mitigate the risk of them being mistaken as only architectural features and inadvertently being removed during the life of the building (e.g., during a future renovation). Moreover, they would need to be designed to receive diaphragm and gravity load from the floor, which they have not.

Furthermore, it is evident that the SEOR is aware of what good practices are for designing masonry shear walls, evidenced by the east-to-west spanning wall used in Area E. Unlike the partition masonry walls, that wall a) was constructed such that axial and diaphragm loads from the roof can be transferred to the wall (Figure 23), b) has vertical reinforcement that is well-anchored to develop shear-friction resistance (Figure 11), and c) has a standalone wall footing without a vapor retarder underneath it, thereby increasing the sliding resistance of the wall (Figure 5).

6.6.2. Deficiencies Apart from Member Capacity

During our review of the structure, we identified several deficiencies in the design of the FLPHQ as they relate to structural integrity, serviceability, and durability requirements or best practice.

While the design generally satisfied the structural integrity provisions required by ACI 318-14 and the FBC, the structural system seems to rely on noncontact lap splices of prestressing strand (in the soffit beams) to “shear-friction bars” to provide continuity of bottom reinforcement. This detail is not directly addressed in either code. We acknowledge that this building system is widely used in the South Florida market; however, we recommend that the SEOR provide justification for its use and clarify how they are providing continuity of bottom reinforcement.

While the nominal cover depths shown on the structural drawings satisfy minimum requirements from ACI 318-14, it is not clear from the precast shop drawings that their understanding of the design cover depths complies with ACI 318-14. In some circumstances, the sectional views indicate that the cover to the transverse reinforcement satisfies minimum requirements; however, other views indicate the opposite. It is possible that this inconsistency is merely an error in production of the precast shop drawings; nonetheless, we recommend that the SEOR verifies and documents this.

While our serviceability checks indicate that story drifts and beam deflections were less than code-prescribed maximum limits, the first period of vibration of the FLPHQ differs substantially from what we would expect of a typical building of similar size. Using a database of the fundamental vibration period of buildings containing concrete shear-wall systems, Goel and Chopra[7] reported that the period of buildings containing no more than four stories ranged from 0.13 to 0.60 seconds. The period of the FLPHQ is greater than 0.9 seconds (based on our analysis models, which include the concrete frame and reinforced concrete shear walls, but not the CMU partition walls), which is significantly larger than the values reported by Goel and Chopra and is an indication the that building is ill-configured (i.e., there is insufficient

[7] Goel, R.K., and A.K. Chopra. 1998. “Period Formulas for Concrete Shear Wall Buildings,” *Journal of Structural Engineering*, V. 124, No. 4: 8 pp.

length of reinforced concrete shear walls). The higher period is likely due to a) a shorter length of shear wall in the FLPHQ, and b) the differences in distribution of that wall on plan than in typical buildings of similar size, resulting in increased flexibility. The first mode shape and displaced shape of the structure when subjected to east-west winds indicate that the building is twisting in addition to laterally swaying. As conceptually illustrated in Figure 34, this twisting component of the first mode behavior is being caused by eccentricity between the center of wind loading and center of rigidity. Since there is greater lateral stiffness from the two sets of reinforced concrete core walls on the northern half of the building than the stiffness from the one set of shear walls on the southern half of the building, the center of rigidity is shifted towards the northern half of the building (approximately near Gridline 5). Moreover, since the centroid of east-west wind loading occurs roughly at the mid-length of the building, the wind loading is eccentric with respect to the center of rigidity and produces twisting. While this condition does not violate any code-prescribed performance limitations, it is not standard practice to design structures to perform this way. Furthermore, it is possible that the twisting may result in unusual serviceability-related conditions, since the lateral system of the FLPHQ differs substantially from typical practice of similarly sized buildings. Proposed measures to remediate member capacity deficiencies (discussed later) also rectify this condition.

6.6.3. Deficiencies in Member Capacity

Beams

During our review, we also identified elements that are not code-compliant for various strength limit states. Several beams adjacent to the rooftop mechanical equipment had DCRs in flexure, shear, or combined shear and torsion that exceed unity. For our analysis, we used the 150 psf live load allowance for the mechanical equipment per Sheet HQ-S0-2-02. However, Sheets HQ-S2-2-4C through 4D explicitly show the weight of various pieces of mechanical equipment. Section 1603.1.1 Floor Live Load of the FBC states that "The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas." The SEOR should clarify on the drawings how loading from mechanical equipment was treated, so questions do not arise in the future.

Columns

Our analysis indicated that 24 columns are overstressed for combined axial and flexural loading. However, only 8 of the columns were overstressed by more than 10% (i.e., the demand-to-capacity ratio exceeded 1.10). Although the remaining 16 columns are still overstressed, they are of less concern because a) it is likely that these members can support a relatively small overstress, since the actual material strengths for concrete and steel typically exceed their design values and b) there is the possibility of moment redistribution once a structural member inelastically deforms and loses stiffness. Based on our experience and judgment, while these 16 columns are overstressed from a design perspective, they would still exhibit sufficient axial-flexural strength to meet their intended level of performance. While the FLPHQ is not considered an existing building (but rather new design), codes for existing buildings, including the 2020 Florida Building Code, Existing, 7th Edition (FEBC) include allowances for the gravity load-carrying structural elements (e.g., columns) to undergo up to a 5% increase in design gravity load without being strengthened.

Additionally, based on our analysis, 23 columns were overstressed in one-way shear and 50 did not satisfy an ACI 318-14 requirement that, when the factored shear demand exceeds one-half of the code-calculated shear strength provided by the concrete, a minimum amount of transverse reinforcement spaced no further than one-half of the effective depth of the member shall be provided. The cross section of all these columns was either 24-inch square or 24-inch diameter round. One-half of the effective depth for the 24-inch square column is generally 10.75 inches and one-half of the effective depth of the 24-inch diameter round columns is 9.6 inches (0.8 times the diameter divided by 2, per Section 22.5.2.2 of ACI 318-14). This ACI reinforcement spacing requirement is intended to mitigate the risk of a shear crack propagating at a 45-degree angle without intercepting any transverse reinforcement. Accordingly, the contribution of any transverse reinforcement spaced more widely than one-half of the effective depth is neglected when calculating the one-way shear strength. In contrast, Sheet HQ-S4-2-01 of the structural drawings shows the transverse reinforcement spacing in the reinforced concrete columns to be 12 inches.

While the spacing of reinforcement in these columns does not strictly meet code requirements, we believe that these columns would still perform adequately for the following reasons.

- Due to the axial compression in the columns, it is likely that a shear crack would propagate at an angle greater than 45 degrees to the column cross section and thereby would intercept at least one piece of transverse reinforcement before propagating through the section. This increased crack angle has been observed in load tests of axially loaded members.
- Some technical documents and codes which indicate that transverse reinforcement spaced more widely than one-half of the effective depth can still contribute to the shear strength of the member. Based on the principles of ACI 369.1-22[8], it is expected that the transverse reinforcement in the 50 columns will contribute to the shear strength. Under ACI 369.1-22, the provided amount of transverse reinforcement exceeds the minimum shear steel area requirements of Section 10.6.2.2 of ACI 318-14.

Since a shear crack would likely intercept transverse reinforcement, we believe that the contribution of the transverse reinforcement to the column shear capacity may be reasonably accounted for. For this reason, we do *not* believe it is necessary to provide shear strengthening for either the 23 columns overstressed in one-way shear or the 50 columns which require minimum shear reinforcement.

Shear Walls

Our analyses, which include the concrete frame and reinforced concrete shear walls, but not the CMU partition walls, also indicated that the DCR in one-way shear for a portion of the south elevator shaft reinforced concrete core wall was well in excess of unity. The same core wall was also overstressed in flexure between Levels 1 and 2.

Foundations

Using service-level (allowable strength design) loadings, we found that the bearing stress under the three isolated footings under the columns at Grid Points G/4, H/4 and K/4 exceeded the allowable bearing capacity believed to have been achieved after vibro-compaction. While Nutting indicated that vibro-compaction (vibro-replacement) “typically improves the soils to provide an allowable bearing capacity of

[8] ACI 369.1-22: Seismic Evaluation and Retrofit of Existing Concrete Buildings

6,000 to 8,000 pounds per square foot, depending on equipment power, time, and soil type", the design for the FLPHQ (and our analyses) was based on an assumed value of 7,000 psf. Therefore, unless it can be demonstrated that the allowable bearing capacity after vibro-compaction exceeds 7,000 psf, alteration of the foregoing footings may be required to achieve code-prescribed levels of safety.

Our analyses also indicated that the foundations under two core walls (CF-1 and CF-3) did not have code-prescribed levels of resistance against foundation sliding. Moreover, the overturning resistance of mat footing CF-3 was less than the code-prescribed minimum level, resulting in uplift on the foundation.

Slide Bearings

The service-level stress in the sliding bearings along Gridline F were also found to exceed their allowable manufacturer limit for allowable bearing stress. The SEOR should review and comment on this condition. Additionally, the SEOR should comment on the range of temperature and exposure conditions which that bearing is expected to undergo throughout its life.

6.6.1. Sensitivity of Area E Lateral System to Column Base Fixity

The first mode period and the lateral deflection at the top of the columns for Area E were both very sensitive to the assumed fixity at the base of the columns. The first mode period was also quite long for a one-story building. Additionally, this area responds to lateral load by twisting. None of these behaviors are desirable. However, even with this undesirable behavior, Area E may still function adequately if the building envelope and finish systems can accommodate the type and magnitude of deflection. Additionally, the slide bearings at the expansion joint between Area E and the main building will need to accommodate the peak lateral displacement while maintaining acceptable performance.

Since the lateral deflection results can vary widely from column to column and are very sensitive to the column base fixity assumptions, we believe that the SEOR should provide the following for review.

1. The peak lateral deflections at the top of the columns;
2. The relative difference in peak lateral deflections at the top of the columns, resulting from service-level and factored-level wind loads;
3. Their assumptions regarding footing fixity which they used to design the FLPHQ;
4. Their assumptions regarding diaphgram rigidity (rigid or semi rigid) and lateral loading applied to the building;
5. The values for all structural member stiffness modifiers (columns, beams, joists, slabs, and walls) which they used to design Area E; and
6. The value for modulus of subgrade reaction which Nutting approved for design of the FLPHQ (with accompanying documentation).

Furthermore, the design consultant responsible for the design of the window wall at Area E should confirm that the installed system is able to accommodate these lateral deflections. Additionally, the AOR, AECOM, should confirm that the building finishes can accommodate these lateral deflections (i.e., drifts). The manufacturer for the slide bearings should confirm the maximum amount of displacement that can be tolerated at the slide bearings under factored design loads. If the building cannot accommodate the design-level deflections and displacements, then it will be necessary to rectify the building. The type of rectification may depend on which system (e.g., window wall vs slide bearings) cannot accommodate the expected movement. The rectification could be a structural modification to the building, or it could be a modification to the specific system. We are not proposing rectifications at this time but may propose them to the City after we review the requested items to be presented by the SEOR.

7. CONCEPTUAL RECTIFICATION OPTIONS

Our structural peer review of the FLPHQ revealed several deficiencies in member strengths which we recommend that the Project Team consider for rectification. There exist various approaches to addressing these structural deficiencies. Conceptual descriptions of several of those approaches are described below. When designing the rectification of an overstressed member, the existing stress/strain in the element should be considered. Note that our recommendations are conceptual, do not constitute a design, and are limited to the structure designated in this report.

7.1. Column Strengthening

Previously we recommended that the 8 columns with DCRs exceeding 1.10 for combined axial and flexural loading be strengthened. Many other columns were noted to have DCRs exceeding 1.0 for various reasons. Despite that we believe these other columns will perform sufficiently, they are still noncompliant with the contract requirements. If the goal is to make them comply with the contract requirements, rectification is needed.

7.1.1. Column Enlargement

The cross section of the column can be enlarged to strengthen the column for both combined axial and flexural loading and for shear. Figure 35 conceptually illustrates the enlargement of a reinforced concrete column. Since it will be difficult to anchor new flexural reinforcement at the top and bottom of the columns due to the presence of potentially conflicting reinforcement in the soffit beams, the flexural strength of the column can be enhanced primarily by increasing the effective depth of the section rather than by adding flexural reinforcing steel.

Enlarging the cross section as shown in Figure 35 provides an added benefit of confining the concrete in the original column. When concrete is confined it has a greater strain capacity, thereby increasing the strength and ductility of the column, even without additional anchored vertical reinforcement.

In the column enlargement rectification shown in Figure 35, the perimeter of the existing column is roughened and cleaned to allow for composite action with the enlargement. New shear-friction reinforcement is post-installed into the existing column to enhance interface shear strength between the original column and the enlargement. For columns that are overstressed in axial-flexural loading, the new transverse reinforcement should be spaced closely to provide confinement. For other columns, the new transverse reinforcement need only be spaced more closely than one-half the effective depth of the column. We expect that using No. 4 reinforcing bar ties at this spacing will meet the minimum shear reinforcement provisions in ACI 318-14.

7.1.2. FRP Jacketing

Fiber-reinforced polymer (FRP) jacketing of the columns may also be a viable option to rectify the axial-flexural and shear conditions previously identified. As illustrated in Figure 36, this rectification entails wrapping a column with FRP and adhering it using epoxy. FPR jacketing can be used to provide confinement to the concrete in the column and provide additional strain capacity as discussed above for the column enlargement. An FRP jacket can also be used to enhance shear resistance and thereby satisfy the code requirement for minimum transverse reinforcement. ACI 440.2-23 (Design and Construction of Externally Bonded Fiber-Reinforced Polymer (FRP) Systems for Strengthening Concrete Structures—Guide) provides guidance for strengthening members with FRP.

7.2. Column Footing Enlargement

The soil bearing pressure was found to exceed the allowable bearing capacity of the soil under the three isolated footings on Gridline 4, between Gridlines G and K. To address this issue, the footing bearing area may be enlarged to distribute the column axial load over a greater area to reduce the average bearing stress in the soil. Composite behavior between the existing footings and the enlargement would need to be ensured by (e.g.,) roughening the surface of the existing footings and providing properly anchored, post-installed reinforcement.

7.3. Beam Flexural Strengthening

Several of the roof beams near the mechanical equipment were found to be overstressed using code-prescribed factored loading and sectional capacities. As noted earlier, depending on the SEOR's assumption regarding loading (specifically, whether the actual weight of the mechanical equipment or the allowance shown on Sheet HQ-S0-2-02 was used to design the FLPHQ), the beams may not be overstressed. Beams found to be overstressed in shear (with or without torsion) may be rectified by enlarging the beams or providing external FRP (as described earlier for the columns). The following conceptual rectification options for flexural strength are offered for consideration should the SEOR confirm that the live load allowance shown on Sheet HQ-S0-2-02 was used to design the beams. For each option, unless the rectification increases the peak strain capacity of the existing beam (by, e.g., providing confinement), it should be confirmed using moment-curvature analysis that the strengthened beam can achieve its targeted resistance prior to concrete crushing in the extreme compression fiber. Should the load be relieved on the existing beam prior to installing the rectification, this analysis may not be required.

7.3.1. Beam Enlargement

Enlarging the overstressed beams can enhance their flexural strength, similar to the upturned beams on roof soffit beams RSB 76 and 79. A conceptual illustration of this type of rectification is shown in Figure 37. The tension-face of the existing beam is roughened to allow for sufficient interface-shear strength with the new enlargement. Post-installed U-shaped ties embedded (at least one development length on either side of the interface) are provided to enhance interface-shear strength. Adequately anchored tension reinforcement is provided in the enlargement to increase flexural resistance of the beam.

7.3.2. Near-Surface Mounted Reinforcement

Similar to beam enlargements, a near-surface mounted (NSM) rectification entails providing new longitudinal tension reinforcement to increase the flexural strength of the overstressed beams. As shown in Figure 38, grooves are cut into the existing beam to receive the new reinforcement. The substrate within the groove should be properly prepared to enhance bond strength with new epoxy or cementitious paste which is placed in the groove with the reinforcement. Steel reinforcement may be used, although non-metallic reinforcement or corrosion-resistant metals may be preferable to mitigate the risk of corrosion.

The effects of fire on the strengthened beam should also be considered. ACI 440.2-23 provides guidance for designing NSM rectifications.

7.3.3. Other Externally Bonded Systems

Other externally bonded systems may be utilized to increase the flexural resistance of a beam. ACI 440.2-23 also provides guidance for designing other external FRP systems, such as bonded laminates (Figure 39). Otherwise, a steel plate may be bonded to the beam but would likely require fireproofing. No matter the system used, sufficient anchorage of the external system should be provided such that it may develop its intended strength at the critical regions requiring strengthening.

7.4. Addition of Reinforced Concrete Shear Wall

Our analyses indicated that a portion of the south elevator shaft walls would be overstressed, if subjected to code-prescribed factored lateral loading. To address this deficiency in the design of the FLPHQ, we recommend that a new reinforced concrete shear wall (and foundation) be added to the structure to reduce the loading demands on the other walls. The location of this wall should be coordinated with the City. This shear wall should span east-west and be located on the southern half of the building to more closely align the center of rigidity of the structure with the assumed center of code-prescribed east-west wind loading. If properly located, the addition of this shear wall will also reduce the twisting response of the building under east-west wind loading. One possible location for a new shear wall is shown in Figure 40. If this location of shear wall is selected (on Gridline 14 between Gridlines H and J) and designed appropriately, it should resolve the sliding and overturning deficiencies of mat footing CF-3, since the addition of the new shear wall would reduce the demand on the core wall above CF-3.

7.5. Addition of Tie Footings at the North Stair Shaft

The addition of a shear wall at the location recommended in Figure 40 will not fully rectify the sliding deficiency of mat footing CF-1 (under the north stair shaft). One way of enhancing the sliding resistance of this footing is to connect or tie it to adjacent column spread footings, as illustrated in Figure 41. If CF-1 is connected to the column spread footings at Grid Points J/3, K/3, and K/4, the additional gravity load on these columns will provide additional frictional resistance against sliding resistance for the combined foundations. To connect the footings, new segments of reinforced concrete foundations can be cast. These new tie footings sections should have sufficient strength to transfer the lateral sliding loads from the mat footing and column spread footings. The new tie footings would also need to be reinforced to preclude other limit states such as (e.g.,) one-way shear.

7.6. Replacement of Slide Bearings

To rectify the overstressed slide bearings, it is recommended that they be replaced with higher-capacity slide bearings. Con-Serv, the manufacturer of the slide bearings used in the FLPHQ, has similar systems with higher bearing capacities which may be viable replacements. The new slide bearing should be selected to have sufficient strength over the range of temperatures and exposure conditions unique to the FLPHQ. Furthermore, it should have sufficient deformability to allow for the wind and thermal induced movement between Area E and the remainder of the building.

The beams above the bearings may be lifted or "jacked" slightly to permit removal of the existing bearings. Care should be taken not to damage the surrounding elements while lifting the beams. If the beam cannot be lifted without damage to surrounding elements, then the beams can be supported on shoring, the near-surface layer of concrete can be removed from the stub columns, and the embedded plate (shown on Detail 15 of Sheet HQ-S4-3-03) can be removed to enable removal of the pads. The stub column should then be repaired before replacing the slide bearings and removing the shoring.

8. LIMITATIONS OF PEER REVIEW

Based on the limited structural peer review conducted by WJE, several conditions have been identified which may be unique, or in some cases, may apply to multiple locations in the FLPHQ. WJE recommends that this report be shared with the Project Team for their consideration and action. As the licensed design professional responsible for the structure, the SEOR (TT) should make any necessary rectifications. Updated drawings, calculations, and other documentation should be provided for review, coordination, approval, and record purposes.

The peer review services provided by WJE have been intended to call attention to areas of ambiguity, possible deficiency, or other anomalies that were identified during a limited review of the available documents. Our review relied on the documents shared with us at the time of our peer review and inaccuracies in those documents may be reflected in our conclusions.

Conditions may exist or develop over time which were either unknown at the time of our review or not shared with us. WJE reserves the right to modify our findings should additional information be made known or become available to us. Our rectification recommendations are conceptual, do not constitute a design, and are limited to the areas and structural elements designated in this report.

The services provided by WJE should be viewed in a proper context and not be construed as replacing or otherwise altering the contractual responsibilities of the Project Team as they relate to the design and construction of the FLPHQ. Although we have endeavored to identify areas of concern, our scope of services has not included an exhaustive or minutely detailed analysis of each design, component, or system specified on the drawings. Accordingly, the responsibility for a proper design remains solely with the design professional whose seal appears on the drawings.

9. FIGURES

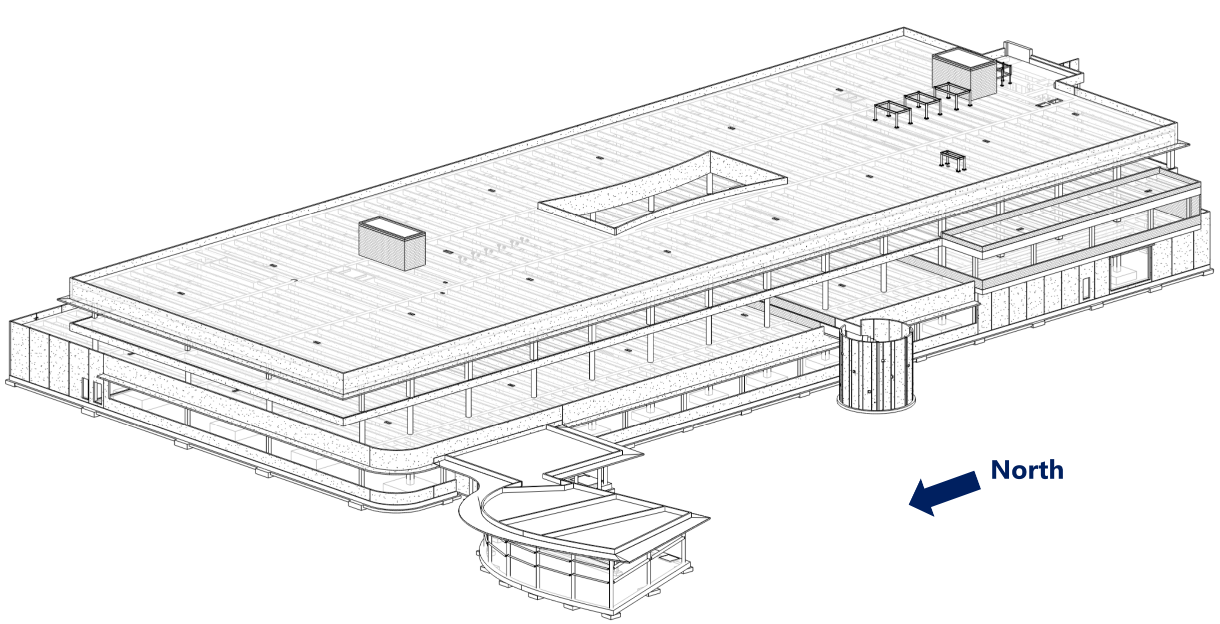


Figure 1. Isometric view of the Fort Lauderdale Police Headquarters building from Sheet HQ-S0-1-00 (dated June 10, 2022) of the structural design drawings. Annotations in blue by WJE

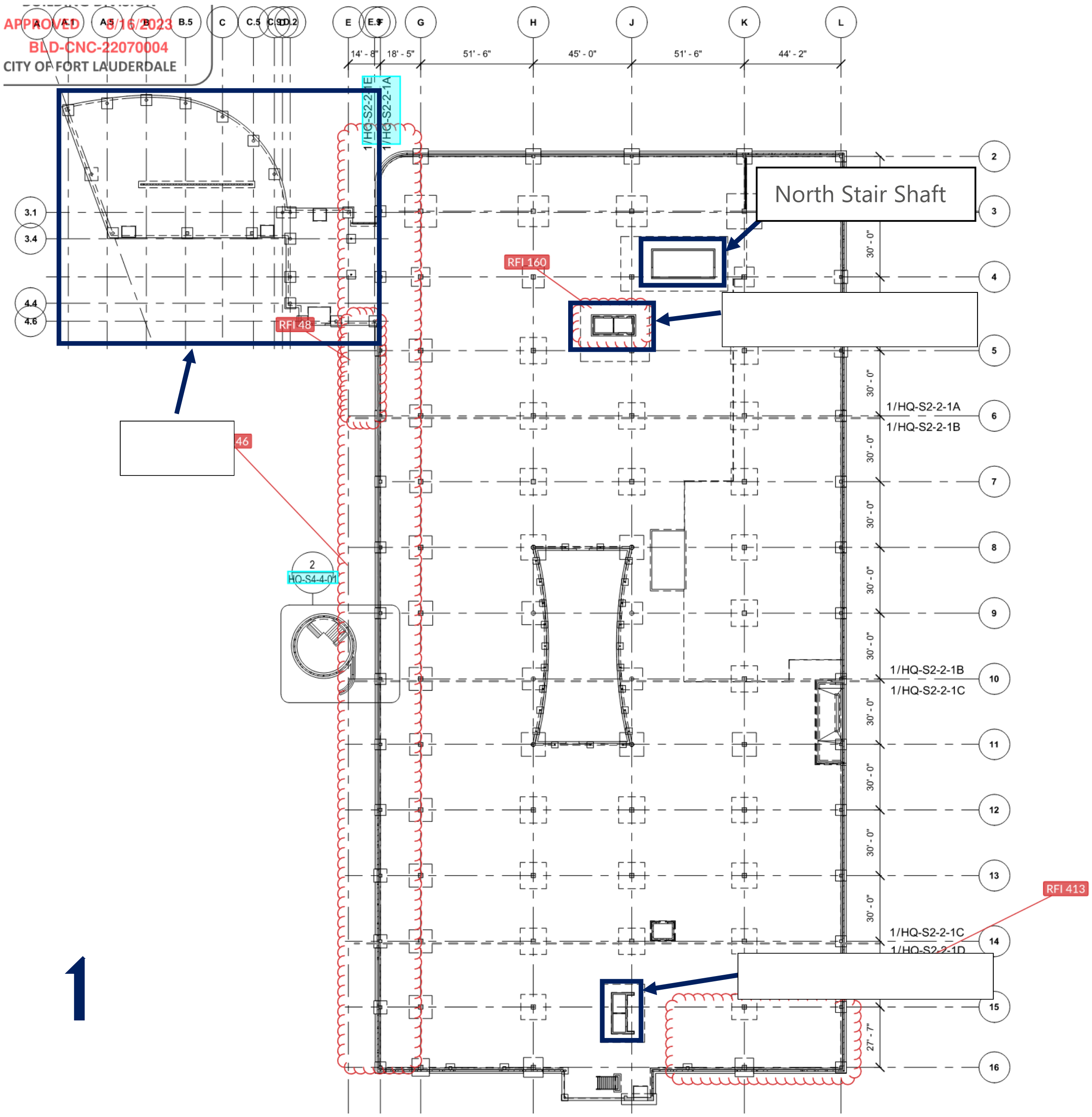


Figure 2. Plan view of the foundation level of the FLPHQ from Sheet HQ-S2-2-01 (dated June 10, 2022) of the structural design drawings with gridlines and shear wall locations shown. Annotations in blue by WJE

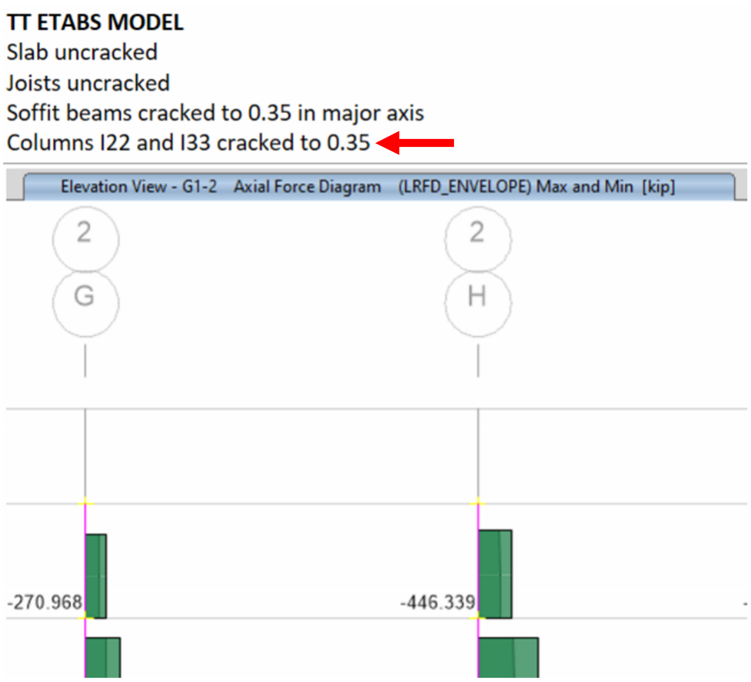


Figure 3. Image from the file "2025311_FLPH Column H-2 and J-2 Validation.pdf" (dated March 11, 2025) prepared by the SEOR which indicates that they used a flexural stiffness modifier of 0.35 for reinforced concrete columns along Gridline 2

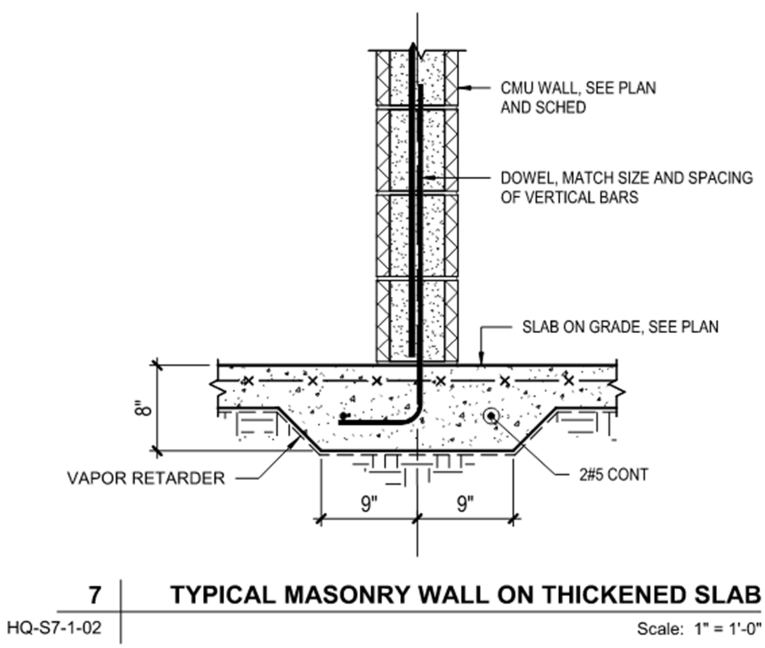


Figure 4. Detail 7 from Sheet HQ-S7-1-02 which shows CMU wall bearing on thickened slab-on-ground. Note that there is a vapor retarder under the slab-on-ground supporting the partition masonry walls

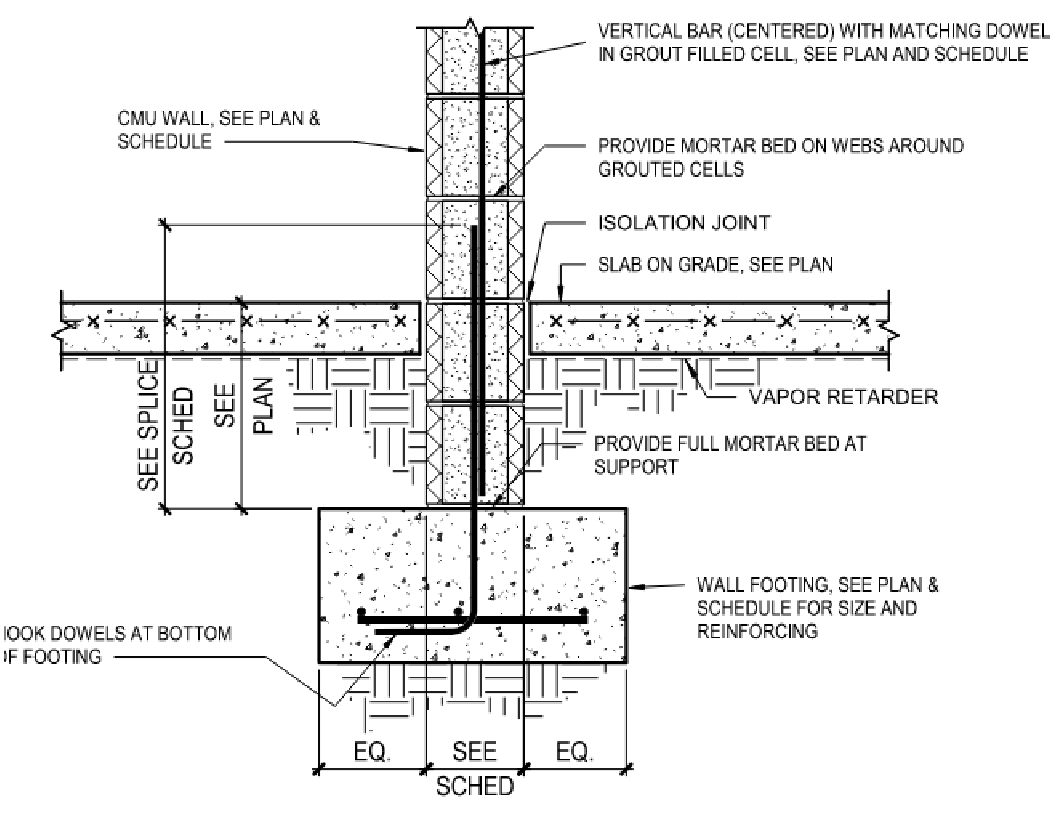


Figure 5. Detail 2 from Sheet HQ-S6-1-03 which shows CMU wall foundation for wall footings that have WF# designation on the foundation plans. Note that the vapor retarder under the slab-on-ground does not pass under the wall footing

WALL SCHEDULE

MARK	TYPE	THICKNESS	REINFORCEMENT		REMARKS
			VERTICAL BARS	HORIZONTAL BARS	
MW8A	CMU	8"	#5 @ 8"	TYP. JT. REINF. AT 16"	GROUT WALL SOLID; TYPICAL AT HOLDING CELLS
MW8	CMU	8"	#5 @ 8"	TYP. JT. REINF. AT 16"	
MW16	CMU	16"	#5 @ 16"	TYP. JT. REINF. AT 16"	
MW24	CMU	24"	#5 @ 24"	TYP. JT. REINF. AT 16"	
MW48	CMU	48"	#5 @ 48"	TYP. JT. REINF. AT 16"	TYPICAL INTERIOR WALL WHEN NO CALLOUT SHOWN ON PLAN
CW8-16	CONC	8"	#5 @ 12"	#5 @ 12"	

Figure 6. Wall schedule from sheet HQ-S6-1-01 showing joint reinforcement for CMU walls. Note that in the schedule that there is no designation or indication of which walls are bearing walls. Also note that the wall thickness column seems to be referring to the spacing of the vertical reinforcement rather than the thickness of the wall

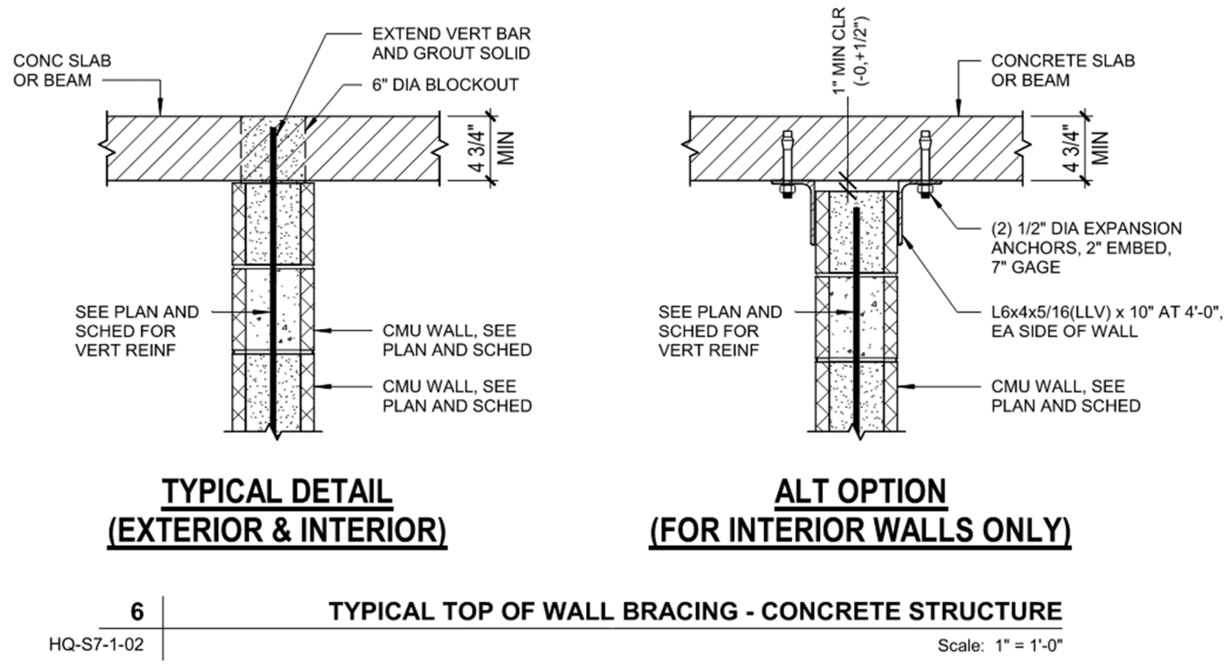


Figure 7. Detail 6 of Sheet HQ-S7-1-02 which shows two options for connecting the partition CMU walls to the elevated slabs or beams

C. Top of Wall Anchors:

1. All anchor components, inserts, dovetail slots and post installed anchors to be hot-dipped galvanized or stainless steel
2. Acceptable Products:

   a) For interior, non-load bearing partitions in Seismic Design Categories A and B only:

      i. PTA Series by Blok-Lok
      ii. PTA Series by Hohmann and Barnard
      iii. At Post-Tensioned Slabs provide dovetail slots or 3/4" (19.1 mm) maximum depth post-installed anchors

   b) For interior, non-load bearing partitions or exterior back-up walls in Seismic Design Categories A and B only:

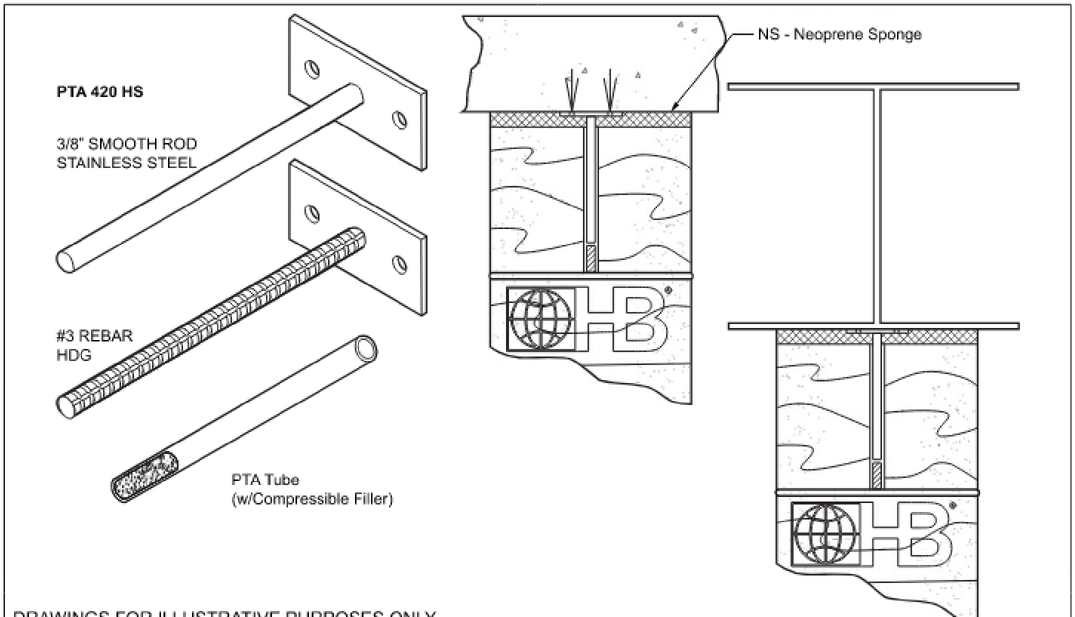
      i. LSA Series by Blok-Lok
      ii. At Post-Tensioned Slabs provide dovetail slots or 3/4" (19.1 mm) maximum depth post-installed anchors

Figure 8. Specification Section 042200 Part 2.6.C with provisions for top of wall anchors in CMU walls

HOHMANN & BARNARD, INC.
a MiTek company

PTA Series Partition Top Anchors

PTA 420 HS



DRAWINGS FOR ILLUSTRATIVE PURPOSES ONLY

PTA 420 HS - PTA Series Partition Top Anchors have been developed to provide lateral shear resistance at the upper limit of masonry walls. They permit vertical deflection of the slab above, without transferring compressive loads to the masonry wall below. PTA Series Anchors are suitable for contruction using steel or concrete. PTA Tube with expansion filler is placed over rod anchor, which has been attached to concrete or steel by any of the methods illustrated. The vertical joint is then filled with mortar, fully surrounding the tube.

Dimensions:
#3 rebar (HOT-DIP GALV.), ⅜" smooth rod (STAINLESS STEEL)
³⁄₁₆" gauge plate (5/16"Ø holes).

Other diameters and gauges available, including heavy-duty styles for hurricane velocity wind loads.

Finishes:
Base Plate (Carbon Steel): ASTM A36
Base Plate (Stainless Steel): ASTM A 666, ASTM A480/480M, and ASTM A240/A240M
AISI Type 304 or 316
Sheet metal (Carbon-Steel):
Hot-Dip Galvanized - ASTM A153/A153M-B2 class B (sheet metal ties and anchors hot-dip galvanized after fabrication).
Note: H&B will certify to a minimum zinc coating of 2.0 oz./ft.²
Rod (#3 rebar): Hot-Dip Galvanized - ASTM A153/A153M-B2
Rod (3/8" smooth rod): Stainless Steel Type 304 - ASTM A582/A582M
PTA Tube: Manufactured from Clear Butyrate.
Tested in conformance with ASTM D542, ASTM D149, ASTM D696 and ASTM D257.

Finishes:
☐ Hot-Dip Galv. Stainless Steel: ☐ Type 304 ☐ Type 316

Note: H&B recommends Stainless Steel for maximum protection against corrosion.

Note: NS Neoprene Sponge also available. Submittal sheet available for download from h-b.com or upon request.

IMPORTANT: Since each construction project is unique, the appropriate selection and use of any product contained herein must be determined by competent architects, engineers and other appropriate professionals who are familiar with the specific requirements of the project in question.
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PRINT FORM
RESET FORM SAVE FORM
SAVE FORM LOCK & HIDE BUTTONS

Figure 9. Hohmann and Barnard specification form for PTA 420 HS anchors for CMU top of wall connections. Form from https://www.h-b.com/products/pta-series-anchors-pta-420-hs (accessed on April 13, 2025)

Figure 10. Rendering of installed PTA 420 HS anchor from Hohmann and Barnard. Image taken from https://www.h-b.com/products/pta-series-anchors-pta-420-hs (accessed on April 13, 2025). Note that the length of the top plate of the anchor is oriented transverse to the length of the wall

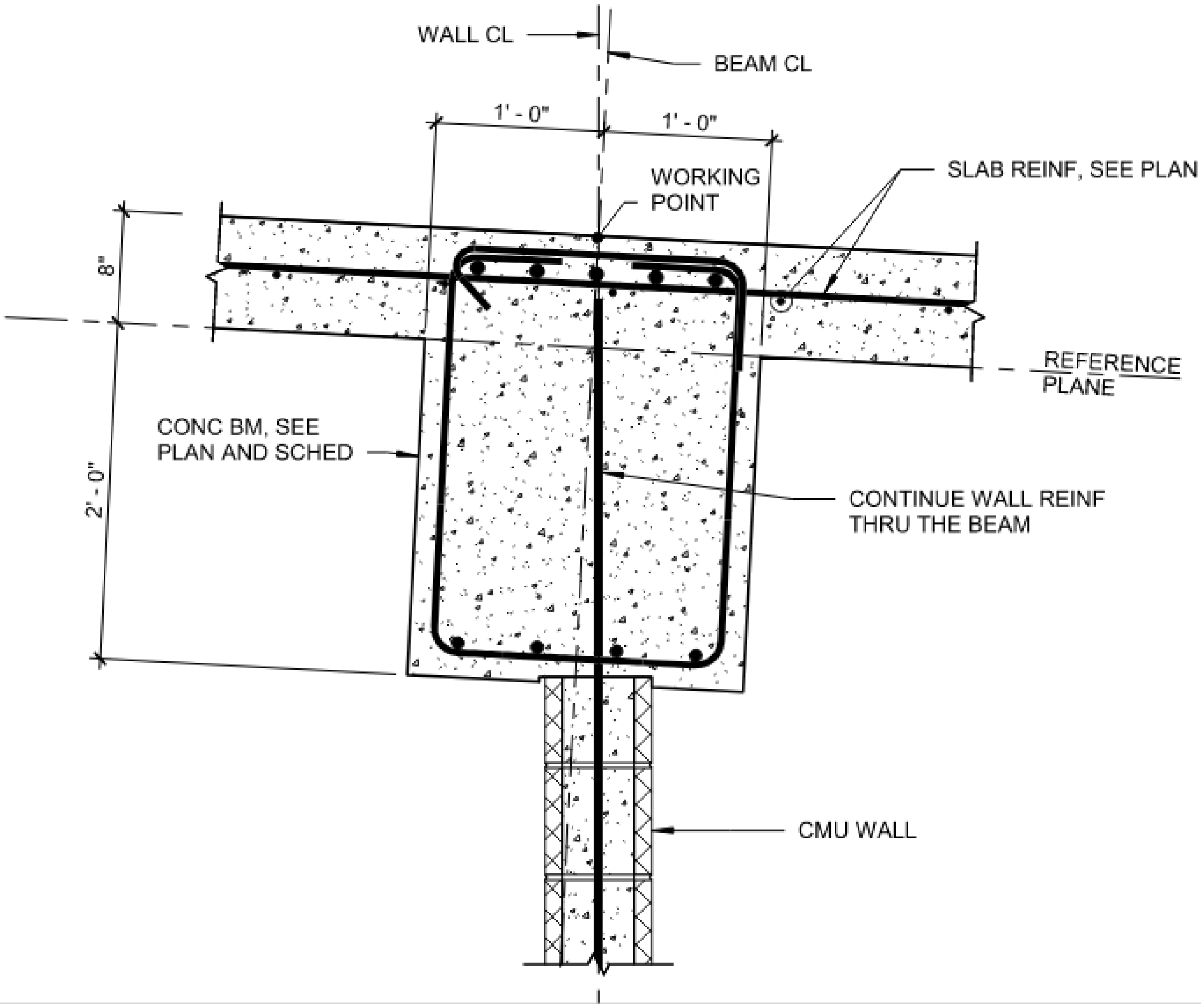


Figure 11. Detail 3 from Sheet HQ-S4-3-04, dated November 15, 2023 showing connection between top of CMU wall and reinforced concrete beam overhead in Area E. Note that the vertical wall reinforcement is well-anchored above the CMU wall to permit shear forces from the diaphragm to be transferred into the wall through shear friction

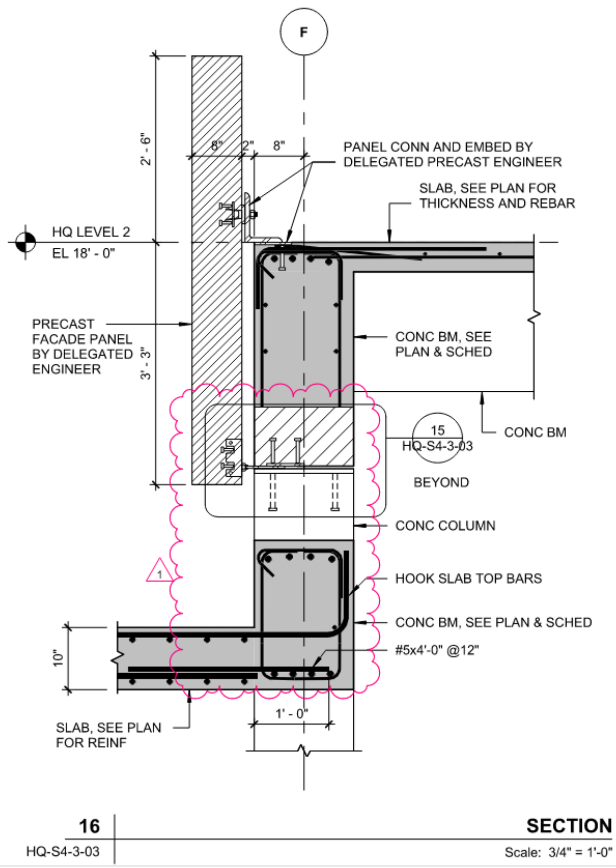


Figure 12. Section through joint between Area E and the main building of the FLPHQ showing the lower beam from Area A the stub column above, and the upper beam from the main building of the FLPHQ. The slide bearing is located at the top of the stub column

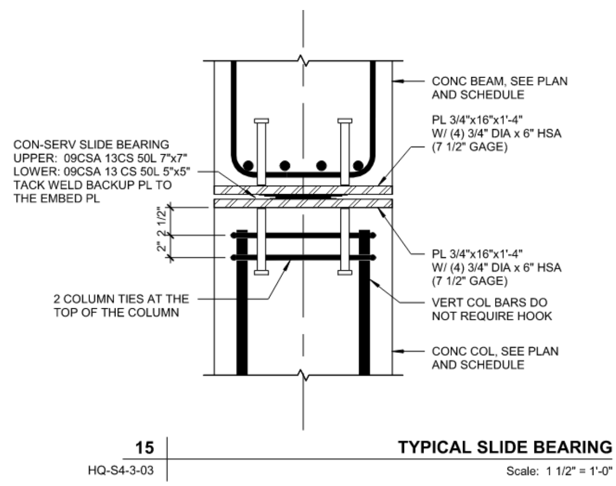


Figure 13. Detail of slide bearing noting the manufacturer (Con-Serv) and the size of the lower portion of the bearing (5 inches by 5 inches)

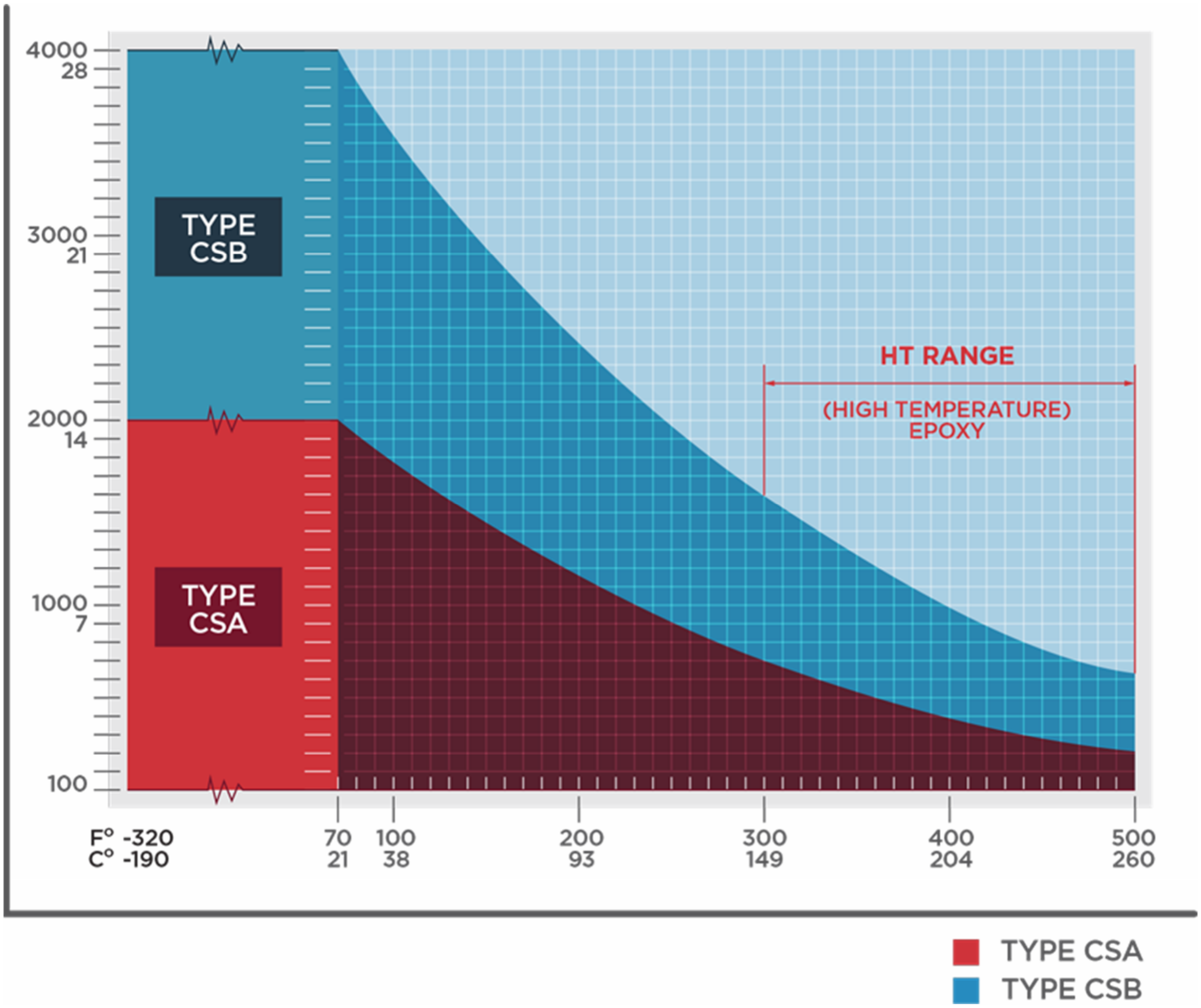
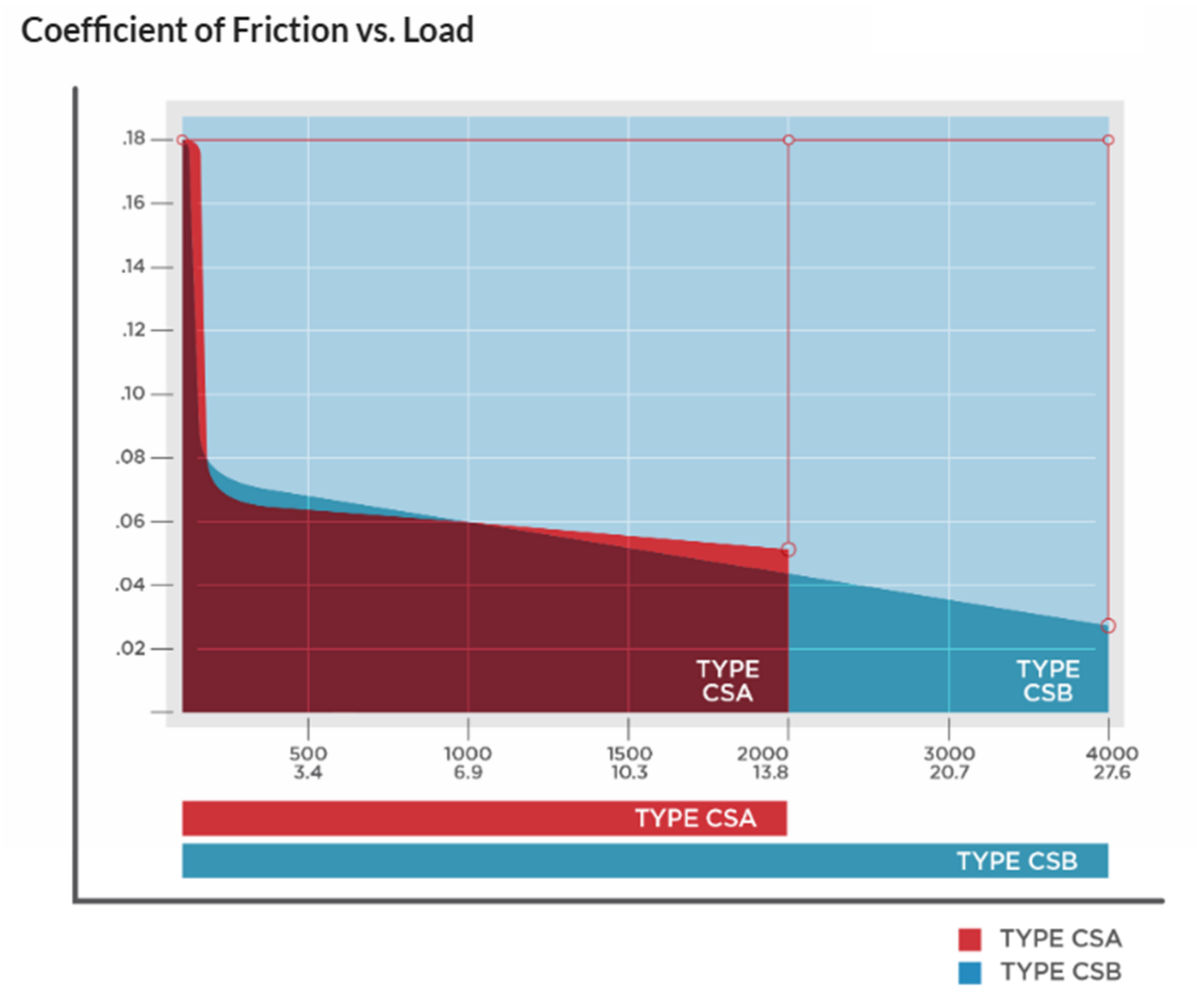


Figure 14. Plot from Con-Serv showing allowable bearing capacity versus temperature. Plot is taken from https://con-servinc.com/con-slide-bearings-type-csa-csb/ (accessed on April 10, 2025)



Coefficient of Friction

The coefficient of friction plotted is a maximum value after first movement breakaway. Friction values will rise with increased speed. The graph values will increase approximately 45% for a speed increase to 10 in./min.

Figure 15. Plot from Con-Serv showing sliding friction coefficient versus bearing stress. Plot is taken from https://con-servinc.com/con-slide-bearings-type-csa-csb/ (accessed on April 10, 2025)

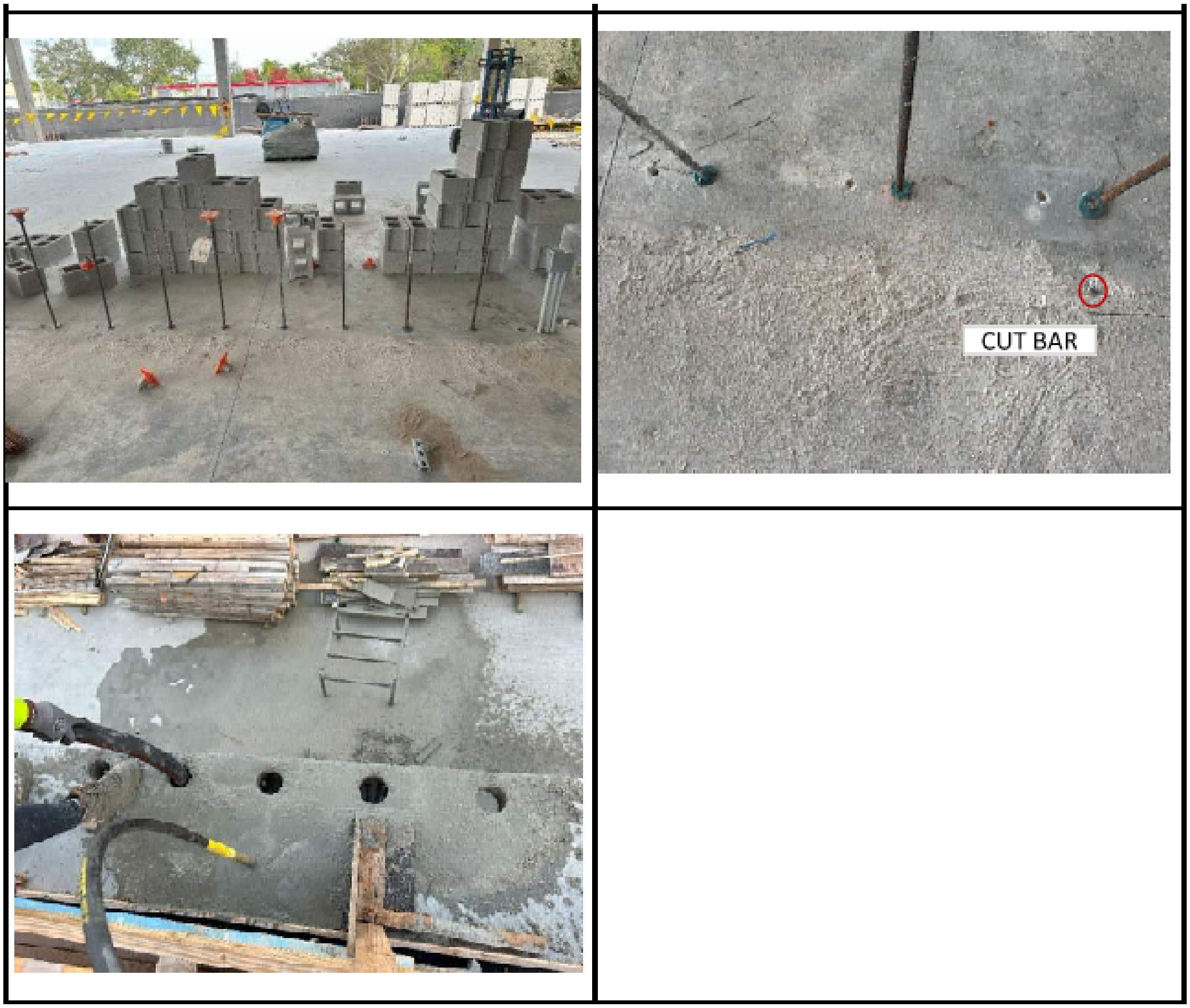


Figure 16. Photographs from Threshold Inspection Report No. 073 showing bars doweled 3 inches into slab-on-ground after original bars were mislocated (top photographs), and filling of grouted cells (bottom photograph)

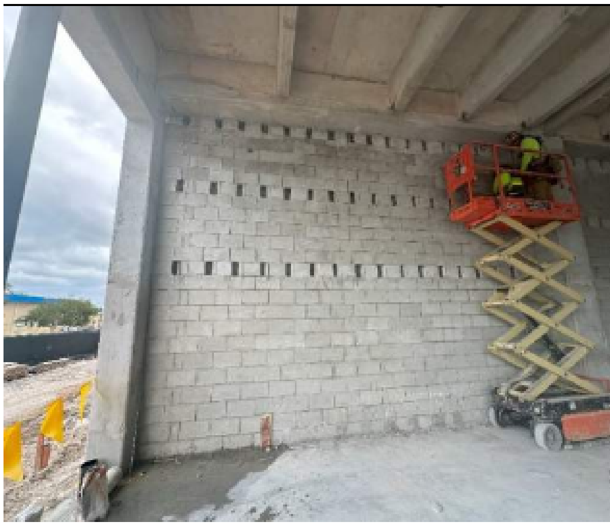



Figure 17. Photographs from Threshold Inspection Report No. 081. Left photograph shows overall wall construction on first story at one location. Right photograph shows the PTA anchors at the top of the wall. Note that two screws are used to connect each anchor to the above element

Figure 18. Photographs from Threshold Inspection Report No. 084. Upper photographs and lower right photograph appear to be of the first story prior to full installation of CMU walls. Note the lack of shoring indicating that the CMU walls were not constructed as bearing walls

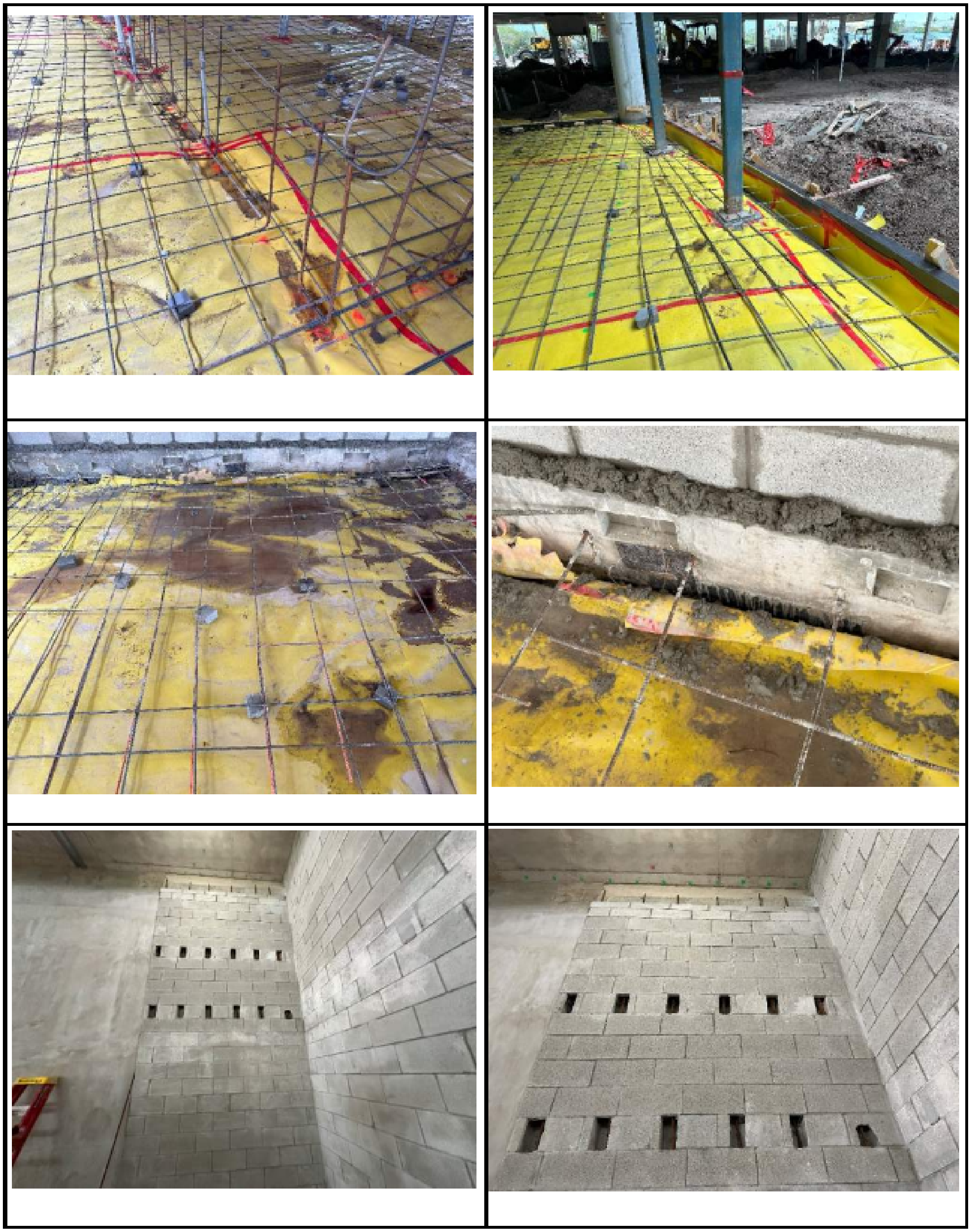


Figure 19. Photographs from Threshold Inspection Report No. 087 showing issue noted by inspector at thickened slab under CMU wall in upper left photograph and reinforcement not extending to the bottom of the slab above in the bottom two photographs

Figure 20. Photographs from Threshold Inspection Report No. 099 showing issue noted by inspector at top of "HQ Level 3 South CMU wall". Note PTA anchors and connection of vertical masonry bars to above slab in left image. Note that the bars are generally spliced to anchors attached to the slab with screws

Figure 21. Photographs from Threshold Inspection Report No. 101. While not clearly stated in the report, these images are believed to be of the third floor slab

Figure 22. Photographs from Threshold Inspection Report No. 103 showing first story CMU walls under construction. Note the lack of shoring indicating that the CMU walls were not constructed as bearing walls

Draft Report

Figure 23. Photographs from Threshold Inspection Report No. 105 showing CMU wall in Area E under construction. Since the wall is being constructed prior to the roof structure, this CMU wall will take load from the roof structure as a bearing wall

Figure 24. Photographs from Threshold Inspection Report No. 107. The photograph on the left shows hooked dowels at a thickened slab location under a wall. The inspector noted that the dowels needed to be shifted downwards. The photograph on the right shows a CMU wall in the first story under construction. Note that no shoring of the structure above is present, indicating that the CMU wall was not constructed as a bearing wall

Draft Report

Figure 25. Photographs from Threshold Inspection Report No. 109 showing dowels extending through cored holes in slab. These dowels appear to be from the CMU wall below

Figure 26. Photographs from Threshold Inspection Report No. 123 showing PTA connectors at bottom of soffit beam with only one screw attaching the plate of the anchor to the soffit beam

Figure 27. Photograph from Threshold Inspection Report No. 125 showing PTA connectors at bottom of soffit beam with the two screws attaching the plate of the anchor to the soffit beam

Figure 28. Photographs from Threshold Inspection Report No. 131 showing a wall that is nearly complete

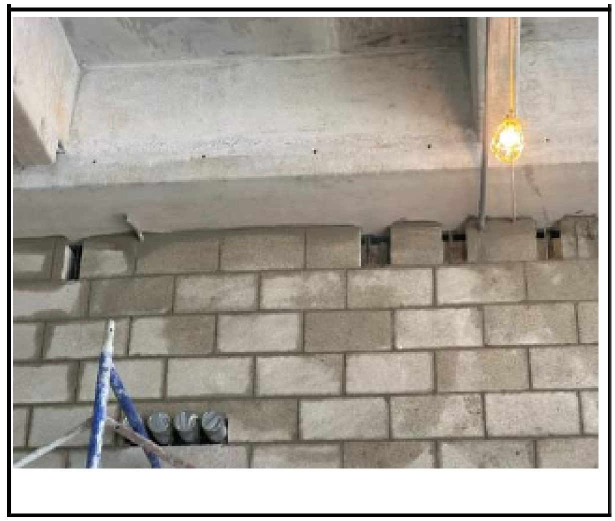
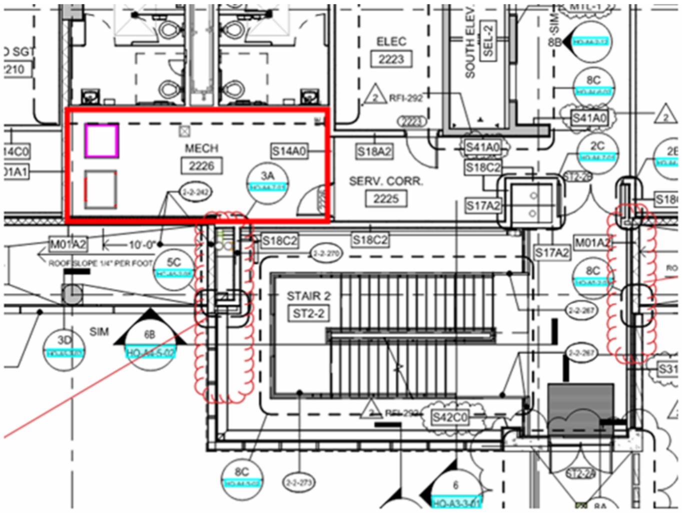
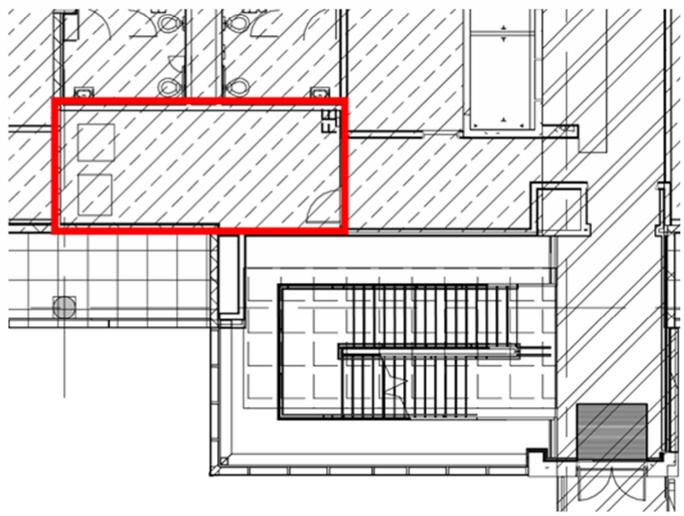


Figure 29. Photographs from Threshold Inspection Report No. 133 showing CMU wall at Level 1, Area D



Sheet HQ-A2-2-2D from Architectural Drawings

MARK	USE	SDL (PSF)	LL (PSF)
	MECHANICAL	15	150
	LIGHT STORAGE	25	125
	LOBBIES	25	100
	EXITS	15	100
	CONFERENCE ROOMS	25	100
	ROOF (OCCUPIED)	40	100
	GENERAL	25	80



Sheet HQ-S0-2-01 from Structural Drawings

Figure 30. Possible inconsistency between architectural drawings (top) and structural drawings (bottom) regarding the location of mechanical equipment. Note that the architectural drawings designate Room 2226 on Level 2 as a mechanical room; however, the structural drawings indicate that a General live load rather than a Mechanical live load was used to design that area

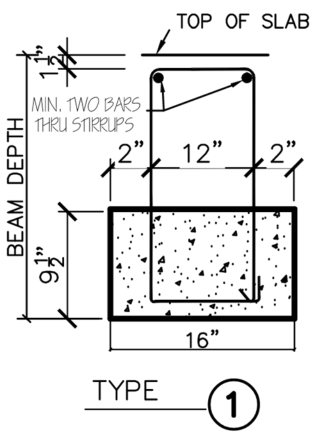


Figure 31. Cross-section of soffit beam (Sheet SC-3C-D of Submittal 328) showing 1.5 inches of cover to the transverse reinforcement

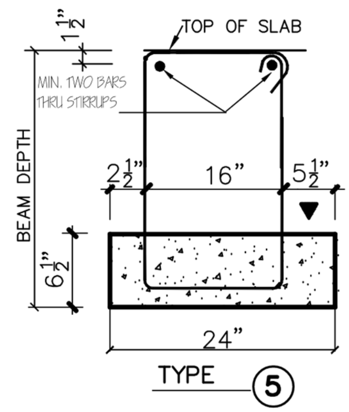


Figure 32. Cross-section of soffit beam (Sheet SC-3C-D of Submittal 328) showing approximately zero inches of cover to the transverse reinforcement

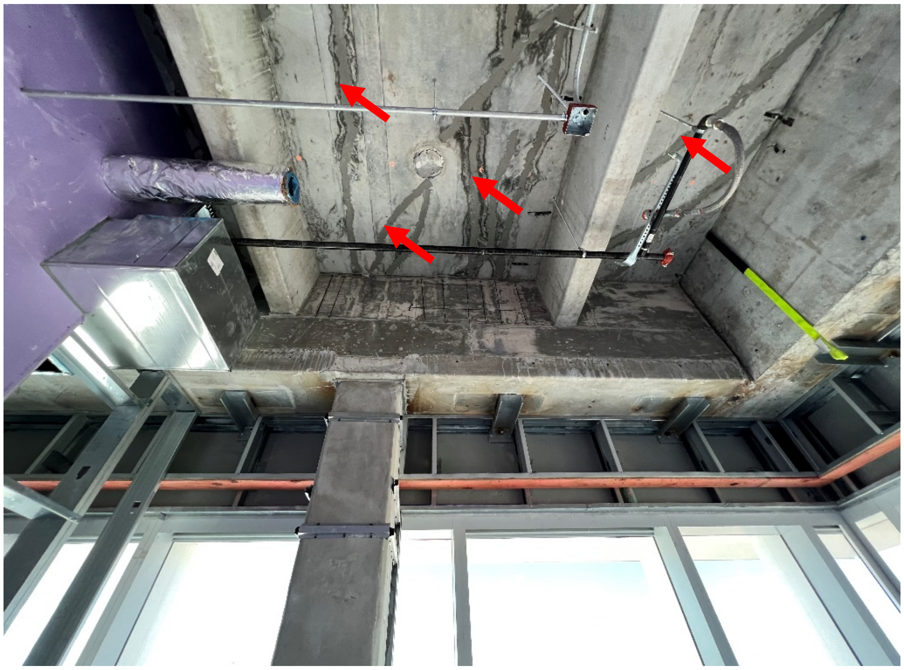


Figure 33. Example of cracks (see red arrows) in the reinforced concrete slab (at the roof) which have been repaired

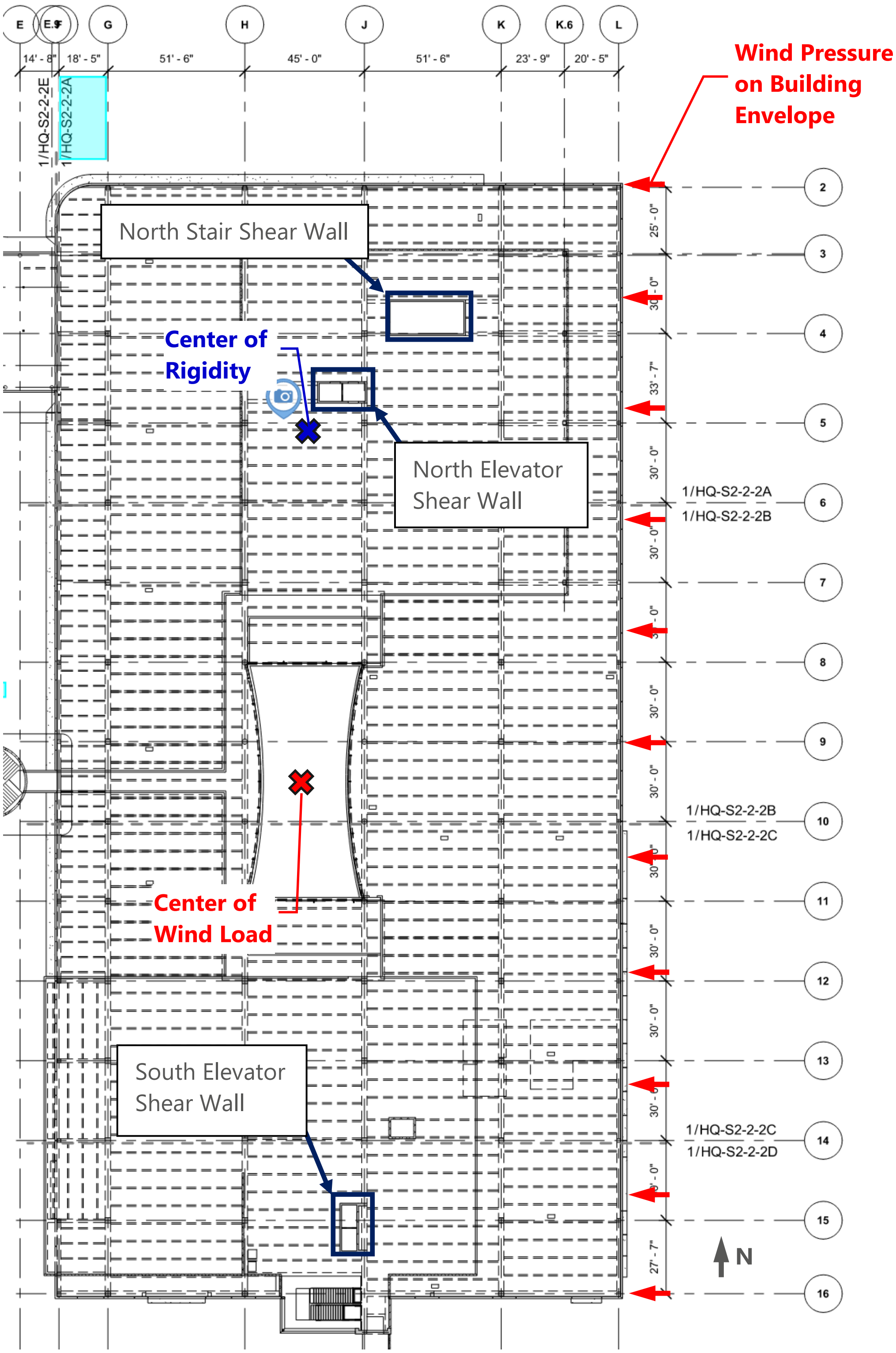


Figure 34. Plan view of FLPHQ showing the approximate locations of the center of rigidity and east-west wind loads. Due to eccentricity between the two, the structure twists

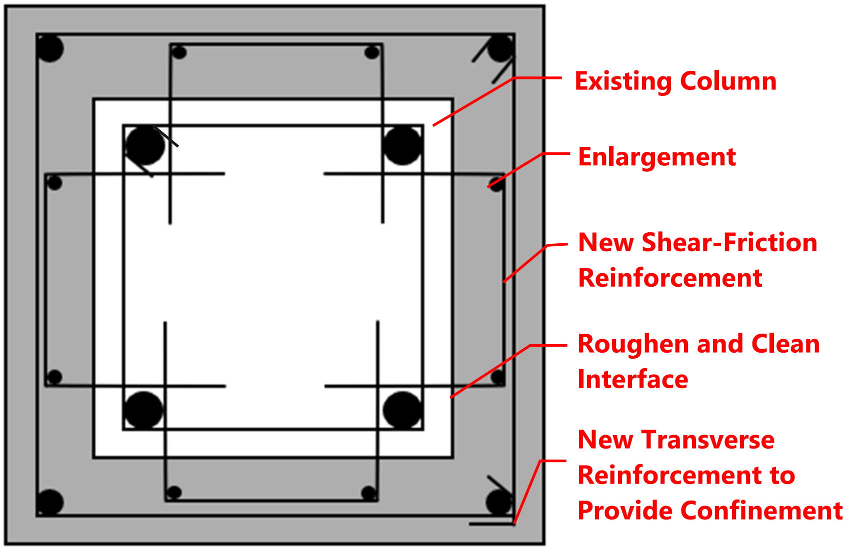


Figure 35. Sectional view of a reinforced concrete column which is enlarged to increase resistance. Note that the new transverse reinforcement should be spaced closely to provide confinement

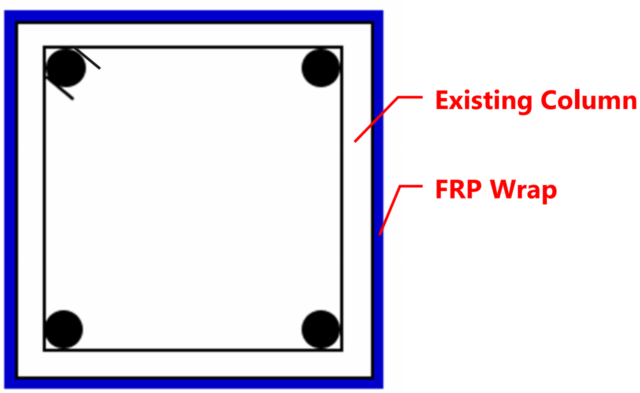


Figure 36. Sectional view of a reinforced concrete column which is wrapped with FRP to enhance shear strength and member deformability

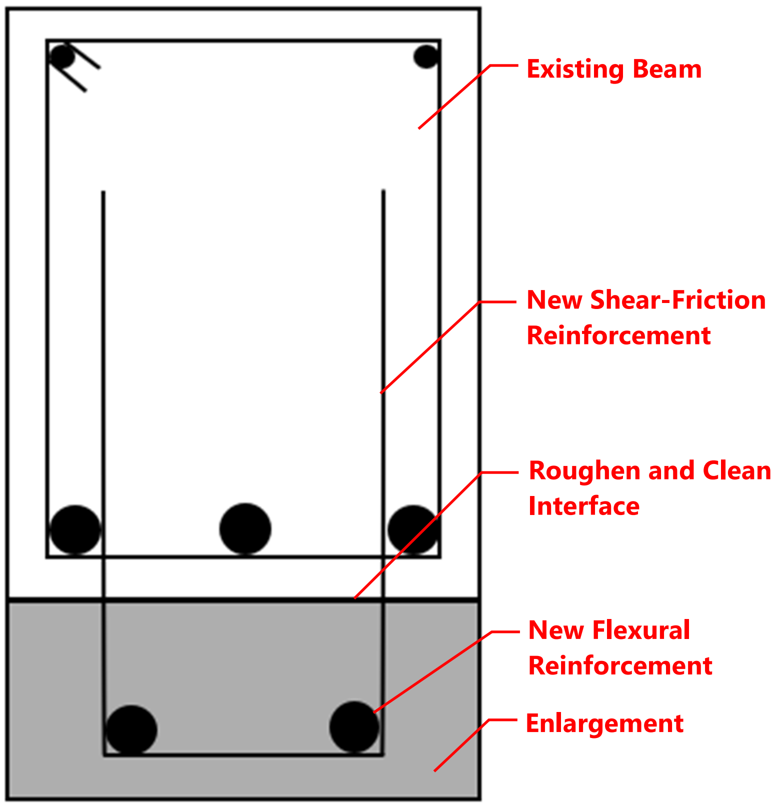


Figure 37. Section view of a reinforced concrete beam which is enlarged to increase flexural resistance

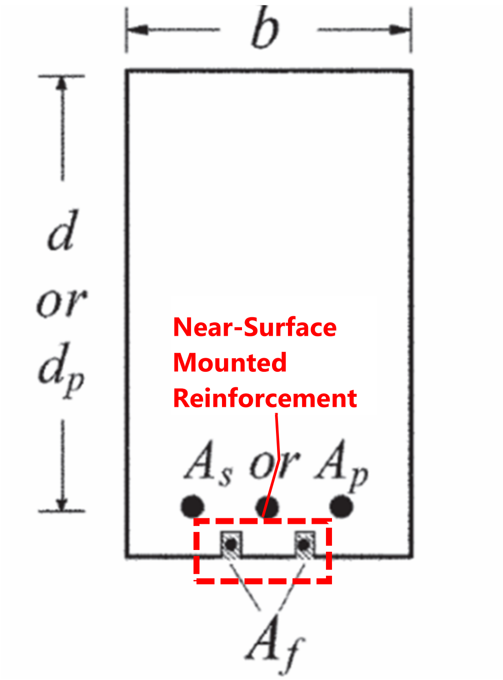


Figure 38. Section view of a reinforced concrete beam showing strengthening with near-surface mounted reinforcement (adapted from ACI 440.2-23)

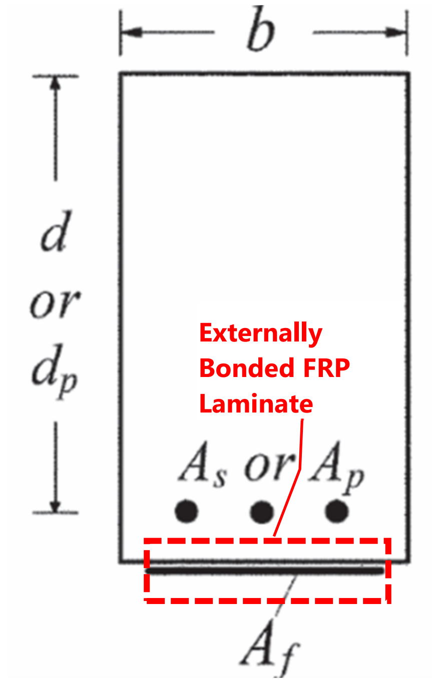


Figure 39. Section view of a reinforced concrete beam strengthened with an externally bonded FRP laminate (adapted from ACI 440.2-23)

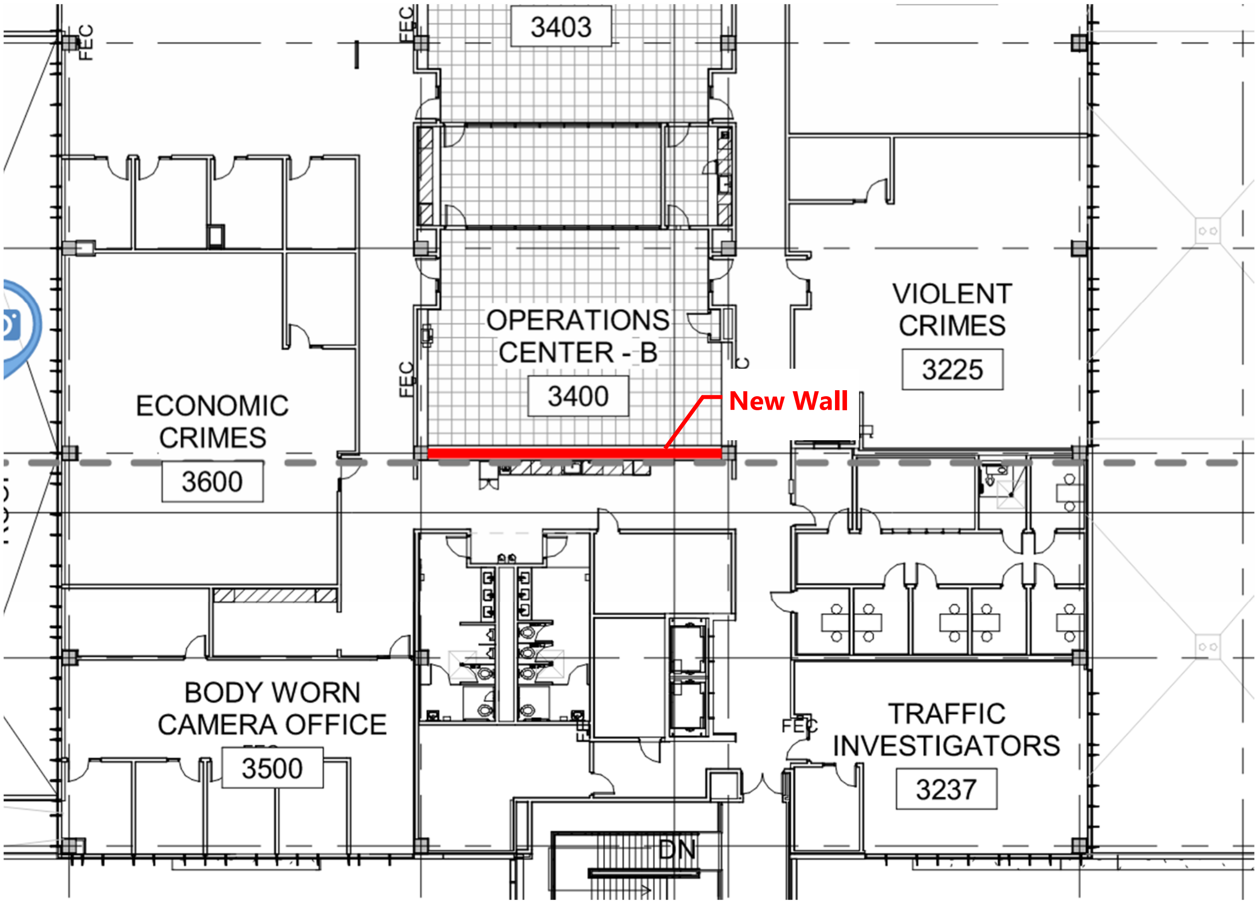


Figure 40. One potential location for the proposed new reinforced concrete shear wall

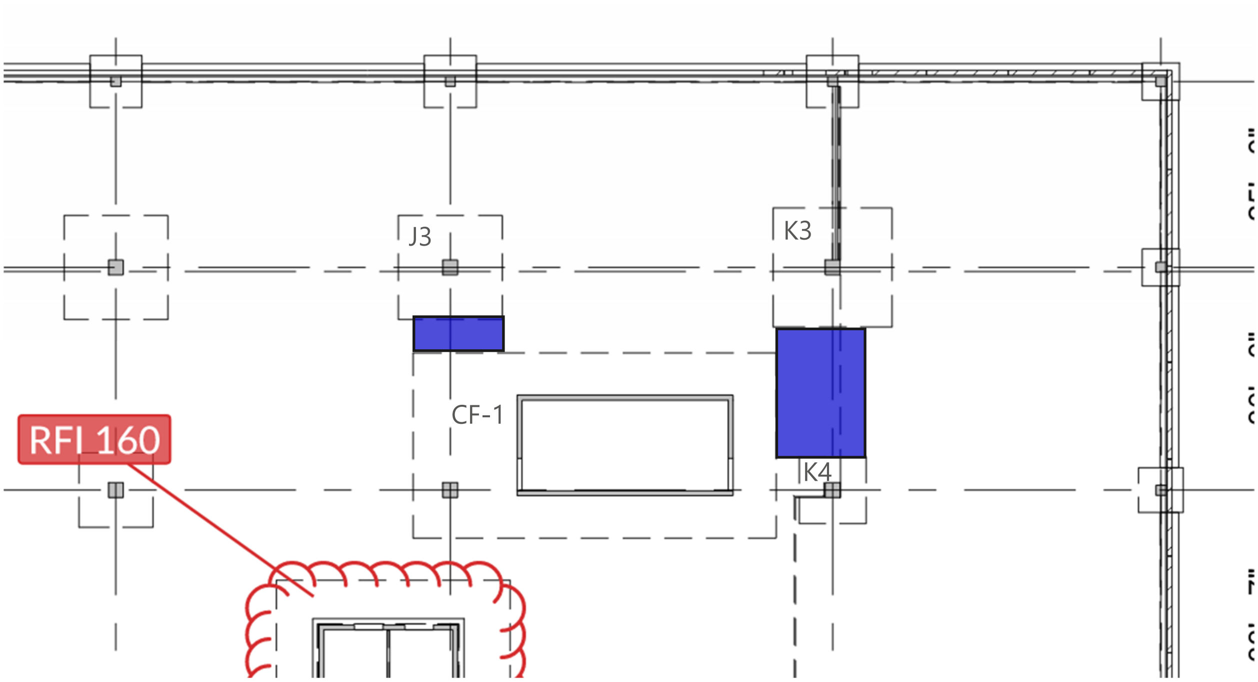


Figure 41. Conceptual rectification of sliding deficiency of mat footing CF-1. The mat footing is attached to nearby isolated footings (J3, K3, and K4) with new reinforced concrete (shown in blue font)